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Development of Sustainable Concretes for the Gulf Region

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Development of Sustainable Concretes for the Gulf Region

By

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DECLARATION

I hereby declare that I am the author of this thesis, that the work recorded has been composed by me, all references cited have been consulted, and that it has not been presented in any previous application for a higher degree.

Nikolaos Strompinis

CERTIFICATE

This is to certify that **Nikolaos Strompinis** has carried out this work under our supervision, and that he has fulfilled Ordinance 39 of the University of Dundee, so that he is qualified to submit the following thesis in application for the degree of Master of Philosophy.

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PUBLICATIONS

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ABSTRACT

Massive construction activities in the state of Qatar have stressed the market of building materials and raised legitimate concerns about the associated impact of construction practices on the environment. As concrete is the most commonly used building material, reducing the eCO_2 of concrete and maximising the use of recycled materials are a necessity for the construction industry to comply with the ambitious sustainability goals of the country.

This thesis describes a 3 year research programme carried out to (i) evaluate the properties of locally produced recycled aggregates, (ii) investigate the properties of a promising non-Portland cement binder and (iii) assess their effect on concrete properties in a performance related approach. The material considered for the study included 2 types of recycled aggregates (CDWA and EWA), two cements (CEM I 52.5N and CSAB 72.5) and two pozzolanic additions (FA-type S and GGBS).

The initial experimental work involved characterization of the test materials. Recycled aggregates were characterized physically and chemically to establish conformity with construction standards. Microstructural, mechanical and durability characteristics of CSAB cement pastes and mortars were investigated in relation to elevated amounts of gypsum aiming to identify an optimum addition. Thereafter, laboratory tests covering fresh, engineering and durability properties were carried out on concrete mixes incorporating various recycled aggregate contents, different binder combinations and encompassing w/c ratios in the range of 0.45 to 0.75. This enabled the measured properties to be also compared on an equal strength basis. Finally, the eCO_2 and the practical implications of these concretes were discussed.

The results from the physical and chemical characterization tests on recycled aggregates suggest that the properties of coarse CDWA and EWA were inferior to that of imported natural aggregates and could not meet all the requirements specified in Qatar construction Specifications (QCS5-2, QS, 2014). Among these, of high importance are the fines contents, water absorption and chemical impurities i.e. chlorides and sulfates. However, the properties were generally found to comply with the broader limits of BS EN 12620 (BSI, 2002) and further processing is required before are used in concrete. Fine recycled aggregates were highly absorptive and rich in sulfates and it is very unlikely to be used in any concrete application.

The study carried out to investigate the properties of CSAB cement paste established that ettringite content increased with increasing the gypsum content and hence it can be used to control the ratio of AFm/AFt hydrates. The lowest paste porosity and permeability was observed for the mixture having 23% w/w gypsum which matches that required to fully convert ye'elimite to ettringite. However, the higher porosity did not correspond to higher strengths. Mixtures with gypsum additions above 2% w/w exhibited lower maximum strengths by approx. 40%. The 2% w/w gypsum addition appeared to be also

critical for setting of cement, above which the setting times became very short. Alongside with ettringite formation, expansion potential was also increased, but above a gypsum content of 9% w/w it dropped back to lower expansion levels. Regarding the chloride ingress, it was concluded that a test of sufficient duration should be used to allow for binding to form and in this case the binding capacity was found to be inversely proportional to the total gypsum content. Similar to other studies, the pH of the cement appeared to be around pH 12.0 which raises concerns about the risk of carbonation induced corrosion.

Tests on recycled aggregate concrete properties showed that given the water absorption of recycled aggregates was treated properly, there was no requirement for higher SP doses as far as the fines content is not too high. Similar to literature findings, the results showed that up to 30% v/v coarse CDWA or EWA incorporation there was no or little influence on the engineering and durability properties of equal w/c ratio concretes. At higher replacement levels though the detrimental effect increased with recycled aggregate content. However, for an equivalent strength, all recycled aggregates performed comparable or even better than reference concrete and displayed additionally lower eCO_2. At a given strength, the addition of pozzolanic materials has also enhanced durability (except carbonation) and sustainability.

Concretes produced with CSABg cement and with or without pozzolanic additions have initially met the target slump but then all mixes lost workability rapidly. Although, as well documented, the early strength development for CSABg-based concretes was faster than that of CEM I-based concretes and achieved maximum strength within 7 days, after 28 days CSABg concretes experience significant strength losses up to 23%. Mixes containing pozzolanic additions at 15% w/w retained their 28-day compressive strength, but no further strength gaining was noted. Given the expansive nature of CSABg-based cements, all concretes exhibited very low drying shrinkages. Similar to CEM I concretes, permeability and durability of CSABg concretes was strongly asssoicated with the w/c ratio of the mix. In comparison with CEM I concretes of equal w/c ratio or strength, CSABg concretes had lower chloride diffusion coefficients, higher resistance to sulfate expansion, lower eCO_2, but higher permeability and reduced carbonation resistance. The latter was due to the low alkaline environment of CSAB cements. As expected, the addition of 15% w/w FA and GGBS in the cement blend has only improved performance at an equal strength basis. In comparison to CEM I/FA or CEM I/GGBS concretes, the engineering, durability and environmental performance of all CSABg-based concretes was comparable, if not inferior.

The practical implications of the study have been considered and overall, it has been shown that locally produced recycled aggregates can be successfully used as a replacement of imported aggregates in concrete applications under a range of exposure conditions. This will yield significant environmental and economic benefits to Qatar. Likewise, CSABg-based cements could be potentially used in concrete applications in aggressive environments and where there is no risk of corrosion induced carbonation. In such cases, the use of CSABg-based cements would satisfy design criteria and reduce eCO_2 of concrete.

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CHAPTER 1: **INTRODUCTION**

1.1 **Field of study**

Over the past decade there has been a great increase in the rate of construction activities in the state of Qatar and the rate is set to increase further in the run up to the FIFA World Cup in 2022. The boom in construction coupled with the requirement to use only premium materials due to the hot and chemically aggressive environment of the Gulf region has stressed the market of building materials and raised legitimate concerns about the associated impact of construction practices on the environment. As Qatar have set ambitious sustainability goals due to its fast growing development, the construction industry is under great pressure to adopt more environmental friendly approaches.

The majority of construction within the Gulf region utilises concrete as the core building material due to wide availability of limestone deposits, the predominant raw material for cement and aggregate production. The impact of concrete production on the environment, however, is a complex mechanism partly governed by the individual impacts from each of these constituent materials and partly governed by the combined effect of these when they are mixed together. Therefore, sustainability issues related to concrete production need to be addressed by considering both the individual and the combined effects of these constituent materials.

Ordinary concrete typically contains 7% to 15 % cement, depending on the performance requirements, 60% to 75% aggregates and 15% to 20% water by mass. Among these, Portland cement (PC) has been always found to be the primary source of CO_2 emissions generated by typical commercially produced concrete mixes, being responsible for almost 85% of total eCO_2 of concrete; approximately 900 kg of CO_2 are emitted to the atmosphere for every tonne of PC clinker produced (CEMBUREAU, 2013; WBCSD and CSI, 2014). A well-established method both to reduce the embodied CO_2 of Portland cement and enhance durability of concrete products is by blending PC clinker with pozzolanic additions materials. Indeed, fly ash (FA) and ground-granulated blast-furnace slag (GGBS) imported from India and Japan, respectively, have been widely used by the cement industry in the Gulf region. If however a substantial reduction in carbon footprint of cement industry is to be achieved a radical change in the way cements are produced is required.

Around the world there are several initiatives working on developing non-Portland cements and alternative binders, with mechanical properties comparable, or superior, to Portland cement and also with smaller carbon footprints. Some potential binders include calcium aluminate cements, calcium sulfoaluminate cements, alkali-activated binders and supersulfated cements (Juenger, et al., 2011). Of the proposed alternative cementitious systems, calcium sulfoaluminate (CSA) cements appeared to be the most promising as they have been shown to have comparable mechanical properties to Portland cement with the added benefits of higher early strength and increased resistance to sulfate attack (Glasser and Zhang, 2001; Telesca, et al., 2014). The latter is of particular interest due to the sulfate rich environment of the Gulf coast. However, CSA cements have not been extensively used in wide scale field conditions and their economic viability and long-term suitability and sustainability has not been fully proven. In addition to that, the huge database of performance information relating to Portland cement cannot be directly applied to this novel binder and a significant amount of work needs to be carried out to establish relationships between composition, mix designs and performance before are used in new construction. Another drawback is the lack of appropriate standards. Currently, the use of CSA cements is only covered by the Chinese specification GB20472 (SAC, 2006), whereas the rest European and American standards consider them as special cements derived from non-Portland clinkers. As a result, CSA cements have limited customer acceptance.

The booming construction industry in Qatar has also stressed the aggregate market. Massive quantities of construction aggregates and sands are required to accommodate the wide expansion of the infrastructure sector, with projections estimating a total aggregate demand of approximatelly 800 million tonnes for the period 2012-2022 (Oxford Business group, 2014). The state of Qatar, however, faces a shortage in construction aggregates. Recourses of quality natural coarse aggregates are limited, as most of local limestone deposits are week, porous and contaminated with sulfates and chlorides (Al-Abidien, 1987). Similarly, reserves of quality concrete sands are diminishing in the region, with the majority of them been either inaccessible or contaminated with gypsum, thus making them unsuitable for use in concrete. Besides that, many of the existing local quality sources will be exhausted in a few years at current rates of extraction. To date, the construction industry relies heavily on importing premium materials, mainly gabbro and limestone aggregates, from neighbouring countries. This gradually resulted in less control over aggregate prices, on the quality of the imported materials, as well as raised

major environmental concerns regarding the CO_2 footprint associated with aggregate transportation.

At the same time, vast quantities of construction and demolition wastes are generating every year from the reconstruction activities taking place all over the country, and are dumped in the dessert causing a significant damage to the environment. The annual production of CDW in Qatar is about 12.5 million tonnes (Oxford Business group, 2014), and concrete products represent a large part of these wastes. If, however, these wastes are recycled and processed into aggregates, they could potentially help to preserve environment and natural resources and reduce dependence on aggregate imports. An additional benefit apart from a resource efficiency and environmental perspective, it that it makes economic sense to turn CDW products into a valuable recourse. This is also in line with Qatar National Vision which aims to ensure that economic growth is balanced with protection of the environment.

Recycled aggregates production from construction and demolition wastes is a well-established industry in many countries and the technology in terms of mechanical crushing and material separation is readily available and relatively inexpensive. In addition to that, the suitability of coarse recycled aggregates for a wide range of applications has been well documented (Dhir, et al., 1999; Sagoe-Crentsil, et al., 2001; Dhir and Paine, 2004; Topçu and Sengel, 2004; Xiao, et al., 2006; Paine and Dhir, 2010) and their use is also covered in national/international standards (ACI 555R: 2001; DIN 4226-100: 2002; BS EN 12620: 2002+A1:2008; BS 8500-2: 2006; JIS A 5021: 2011, JIS A 5022: 2012; JIS A 5023: 2012; BS EN 206: 2013; QCS 5: 2014).

Compared to natural aggregate (NA) the quality of recycled aggregate is considered lower, which restricts its massive use in varieties of construction applications. Customarily, coarse recycled aggregates are mainly used as aggregates in granular base or sub-base applications, for embankment construction and earth construction works and to a less extent in concrete products. Typical replacement levels of virgin aggregates by coarse recycled aggregates account for 20% to 30% in structural concrete (BS EN206: 2013 permits up to 50% depending on the exposure class) with concrete specifiers and producers be reluctant towards greater replacement levels because of the reduced concrete performance. Similarly, fine recycled aggregates are only found limited use in concrete industry because the material is usually contaminated with gypsum plaster known to increase the sulfate content of concrete and cause delayed ettringite formation and its use is not currently covered by the relevant standards.

The standards, however, do not preclude the use of fine recycled aggregates in concrete or the higher substitution of natural coarse aggregates with recycled ones where it is demonstrated that significant quantities of deleterious materials are not present and their suitability for a certain concrete application has been proved. There is a great potential, therefore, for low grade and non-structural concretes to achieve higher replacement levels, even up to 100%, giving to the construction industry the prospect to further reduce the need for primary aggregates and conserve landfill void space in a long term base. While accepting the need to promote the use of recycled aggregates to a greater extent, it must be remembered that aggregates for concrete applications must meet the requirements set in relevant specifications for their particular use.

As the country progresses towards desirable global standards there is a necessity to seek for alternative and more sustainable construction materials, investigate their properties and subsequent behaviour within cementitious systems and finally ensure its suitability as a valuable resource in construction. In addition to that, most of the construction specifications in the Middle East countries are based on international standards i.e. BS-EN and ASTM, which may not consider the local exposure conditions (i.e. extreme temperatures and chemically aggressive environment) and material availability in the Middle East.

1.2 **Aims and Objectives**

To-date, research studies at the University of Dundee corresponding to the incorporation of recycled materials and industrial by-products in concrete have focused on performance related approaches, feasibility opportunities and specification guidance that all facilitate its widespread use and support decision making by specifiers and engineers.

This present project in collaboration with the University of Aberdeen seeks to develop sustainable and environmentally friendly concretes using predominately processed recycled aggregates and a low carbon cement that will be used in the construction of durable, high quality and low carbon building infrastructure projects across Qatar, and potentially in other parts of the world.

In order to meet this, the following objectives were set:

- Determine the physical, chemical and mechanical properties of coarse and fine recycled aggregates sourced from Qatar, establish its variability and evaluate its suitability for use in concrete.
- Study the properties of CSA-belite (CSAB) cement in relation to gypsum content within paste and mortar systems to establish the potential of CSAB cement to produce structural grade concrete.
- Investigate the fresh, engineering and durability properties of CEM I and CSAB concretes incorporating various levels of coarse recycled aggregates and/or pozzolanic additions.
- Evaluate the suitability and sustainability of above materials in a performance related approach and identify the most-appropriate forms of sustainable concrete for use in the Gulf region with regard to the balance between strength, durability, and CO_2 emissions.

1.3 **Scope of investigation**

The scope of the investigation broadly covers the influence of recycled aggregates and an eco-friendly cement on the concrete properties. This involved characterisation of the materials, followed by concrete tests (fresh, mechanical and durability properties). The scope of these is explained in more detail below. The practical implications of using sustainable materials in concrete production were then considered.

1.3.1 Sources and characterisation of materials

The use of recycled aggregates in unbound applications, concrete and concrete blocks was only permitted in latest edition of the Qatar Construction Specifications (5th edition, 2014) and hence the properties of recycled aggregates and their potential effect in properties of locally produced concretes have not been well documented. In this regard, processed recycled aggregates comprising two types (CDWA and EWA) and three size fractions (10/20, 4/10 and 0/4) were obtained from a construction and demolition waste treatment area in Qatar and characterised in accordance with the guidance given in BS EN 12620 (BSI, 2002). This involved testing the physical composition, particle size distribution, particle density and water absorption, particle shape, resistance to fragmentation, drying shrinkage, alkali-silica reaction, LOI, chemical composition and phase composition. Properties such as resistance to freezing and thawing or resistance to surface abrasion where not tested either because were not applicable to the

environment of Qatar or were not agreed during the project planning. The results of the characterisation tests were then analysed to establish variability and conformity with the British and Qatar construction standards. Alongside with recycled aggregates, natural aggregates (NA) which are typically used by the local industry in concrete production have also been included as benchmark materials.

As concrete structures in the Gulf region are exposed to an aggressive environment (e.g. significant temperature variations, chloride and sulfate attack), a CSA-belite (CSAB) cement sourced from China was included in the study with the aim to improve durability, particularly the sulfate resistance, and enhance sustainability of concretes owing to its lower embodied CO_2. The physical and chemical characteristics of material were provided by our collaborative team in University of Aberdeen. Before CSAB been used in concrete, its performance in relation to microstructure, setting time, volume stability and strength development and chloride ingress was investigated in a performance related approach using elevated amounts of added gypsum (required for hydration of CSAB). These properties were considered because they provide information on the gypsum content required for optimum performance.

1.3.2 Concrete testing

Having established the characteristics of the recycled aggregates and the CSAB cement, the properties of concrete utilising them were investigated. The first phase involved testing the fresh (slump), mechanical (strength development, drying shrinkage and water absorption) and durability (chloride ingress, carbonation rates, sulfate expansion) properties of concretes containing RA at replacement levels of natural aggregates up to 100% v/v for 0.45 and 0.55 w/c ratios, and up to 100% v/v for 0.65 and 0.75 w/c ratios. These w/c ratios were selected to cover the full practical range of concretes, to allow equal strength comparisons by interpolation and to cover the requirements/ranges in BS 8500-1 (BSI, 2015). A CEM I (52.5N class) cement conforming to BS EN 197-1 (BSI, 2011) was used in that phase. All mixes were plasticized using a high-range water-reducing admixture conforming to BS EN 934-2 (BSI, 2009). The results of this study were then validated against control mixes prepared with typical coarse and fine construction aggregates from the Gulf region. A type-S fly ash conforming to BS EN 450-1 (BSI, 2012) and a GGBS conforming to BS 15167-1 & 2 (2006) were also considered as a binary cement components at 40% and 50% w/w of cement, respectively, for selective recycled aggregate concrete mixes, aiming to improve durability performance and reduce embodied CO_2.

At phase two, the fresh, mechanical and durability properties (as described above) of concretes incorporating a commercial CSA-belite (CSAB) cement of strength grade 72.5 MPa at 3 days, w/c ratios equal to 0.55, 0.65 and 0.75, a superplasticizer admixture and a fixed recycled aggregate replacement level of 50% v/v were tested. In all mixes a reagent-grade gypsum was added to CSAB cement as required for optimum performance. The effect of 15% w/w FA and GGBS additions to the performance of CSAB concretes containing no recycled aggregates at two w/c ratios equal to 0.45 and 0.55 was also investigated. The range of w/c ratios selected in that phase gave strengths comparable with those of CEM I concretes and enabled the measured properties to be compared on an equal strength basis.

The test results from all concrete mixes were finally compared for equal strength concretes in order to establish whether i) the RA constitutes a valuable resource of aggregates for new construction, (ii) the performance of CSAB concretes is comparable or superior to that of PC concretes, iii) the use of pozzolanic additions can balance the adverse effects of recycled aggregates and enhance the properties of CSAB concretes and to report on the achieved CO_2 savings.

1.4 **Outline of Thesis**

Chapter 2 initially reviews the aggregate market in the state of Qatar and the challenges faced by the construction industry. Following this, the properties of recycled aggregates and how these are covered in national/international standards are analysed. The properties of recycled aggregate concretes are also covered comprehensively. The last part of the review discusses the production process of CSA cement and its variations, the effect of ettringite as a reaction product and the properties of pastes and concretes made from CSA cement in order to build up a better understanding of the material.

Chapter 3 includes the programme of work, materials used and describes the various test methods employed for the material characterisation as well the concrete testing. It also covers information relating to concrete mix design.

Chapter 4 reports on the characterisation results obtained for recycled aggregates and CSAB cement. Initially, the physical, mechanical and chemical properties of recycled aggregates were analysed and discussed. An attempt was also made to find correlations between some of these

properties. The results were then compared against the limits set by British and Qatari construction specifications to establish whether recycled aggregates can be used and to what extent to concrete production. Following aggregate characterisation, the properties of the CSAB cement having various gypsum additions were presented. The relative impact of gypsum content in microstructure, setting times, alkalinity, compressive strength, expansion potential and chloride ingress was analysed, trends were established and an optimum amount was identified for best performance.

Chapter 5 reports on the fresh, engineering and durability properties of concretes containing various recycled aggregates contents. Initially, the properties were compared at an equal w/c ratio to establish the effect of recycled aggregate replacement level. Then, a more practical and useful comparison of concrete properties was made on an equal strength basis, given that compressive strength of concrete is a mix design key criterion for the construction industry. Comparisons were also made with literature findings to determine whether recycled aggregate concretes follow established trends, and with control mixes produced with natural aggregates from the Gulf region to validate the findings against construction practises in Qatar. The effect of FA and GGBS as a binary cement components in recycled aggregate concretes was also reported.

Chapter 6 discusses the findings of the concrete work carried out to evaluate the performance of concretes produced with a CSAB cement and incorporating recycled aggregates or pozzolanic additions. As before, the results were compared for equal w/c ratio and strength to understand the behaviour of these novel concretes.

Chapter 7 compares the performance, including sustainability, of all test concretes on an equal strength basis and addresses the practical implications of the study with regard to the use of recycled aggregates and CSAB cements in concrete applications in the Gulf region.

Chapter 8 presents the overall conclusions of the study and gives recommendations for further work.

CHAPTER 2: **LITERATURE REVIEW**

2.1 **Introduction**

Qatar and other Gulf countries have a hot dessert climate with very low rainfall (75mm per year), dry winds and rapid daily and seasonal temperature and humidity variations. Indeed, the daily relative humidity varies between 56-63% during the summer to 65-95% during the winter period. The daily temperature variations during these periods are 29-43^{o}C and 13-22^{o}C, respectively. These features present a harsh environment for construction, with a number of other challenging features. From these of particular interest are:

- High chloride and sulfate contents in the rock deposits, soil and groundwater, with the potential to cause corrosion or expansive reactions.
- High chloride and sulfate contents in dust in coastal areas, causing attack on the exposed concrete structures.

Concrete structures in such aggressive environment requires advanced design methods, a high level of construction practice and premium raw material to ensure a sufficient service life expectancy. For many years, the construction industry was focused on achieving the required level of concrete durability and serviceability of the structure, without paying great attention on the impact of associated construction approaches on the local environment. This perception of durable but not sustainable structures has now changed, and as the country progresses towards a more desirable building environment, there is a growing interest and support from government, industry and stakeholders to use more low carbon cements and recycled materials in concrete production.

2.2 **Qatar towards a sustainable development**

2.2.1 Introduction

Qatar is a peninsula on the south shore of the Arabian Gulf, with a population of about 2 million. The exploitation of country's natural gas reserves place Qatar among the world's fastest growing economies. Alongside with the economic development, Qatar in its National Vision (QNV) aims by 2030 to be a developed nation capable of providing a high standard of

leaving for all its people, sustaining its development and securing harmony between economic growth, social development and environmental protection (GSDP, 2011).

The government of Qatar under the framework of Qatar National Vision 2030 and with the focus to diversify the country from an oil and gas dependent economy, has invested considerably in developing the country's infrastructures. This led the construction and building industry sector to undergo a rapid growth in the past few years. Recently, Qatar has also been awarded the FIFA 2022 World Cup, which is expected to be a key driver for industry growth, as a massive construction programme of world class sporting venues, international airport and harbour, rail networks and accommodation facilities is expected to be undertaken by the government in preparation for the event. This will also help Qatar to meet the targets it has set itself in QNV.

2.2.2 Aggregate supply in Qatar

The sheer volume of construction projects undertaking in Qatar and particularly in Doha, coupled with rapid speeds of construction activities adopted in order the country to meet World Cup goals have stressed the market of building materials, including the aggregate industry. Massive quantities (>80 million tonnes pa) of qualitatively construction aggregates and sands are required to accommodate the wide expansion of the infrastructure sector.

Indeed, Qatar faces a shortage in natural coarse aggregates and sands. The only available natural coarse aggregates in Qatar are local limestone aggregates, however, these are generally originated from relatively young and week sedimentary rocks and found to exhibit a substantial heterogeneity, particularly in terms of porosity, abrasion resistance and contamination, i.e. chlorides and sulphates (Fookes and Higginbottom, 1980; Al-Abidien, 1987, Fourniadis, 2010; Al-Ansary and Iyengal, 2013). Besides that, most of local limestones are rich in magnesium, and some are true dolomites, which have been known for several years to be liable in alkali-carbonate reaction under the presence of moderate to high sulphate and chloride exposures, or may contain chert or other forms of silica, with the consequent possibility of alkali-silica reaction (Fookes and Higginbottom, 1980). Since concrete structures in Saudi Arabia are exposed to extreme climate conditions, as well as to chemically aggressive environment, especially on the Gulf Coast, and the selection of qualitative aggregates is known to be vital for producing durable concretes, local limestone aggregates find only limited use, mostly in unbound construction applications.

The construction industry for the most part is reliant on imported premium materials, mainly limestone and gabbro (igneous rock – chemically equivalent to basalt) aggregates from the United Arab Emirates (UAE) which meet the physical, mechanical and chemical limits set on construction standards of Qatar. According to Qatar Primary Materials Company aggregate imports in Qatar have been gradually increased over the last years, from approximately 6 Mt in 2005 to 20 Mt in 2012 and remained almost steady at 20 Mt per year between 2012 and 2014 (QPMC, 2013). This steady increase in supply of aggregates is mainly attributed to the delay of letting construction projects and concerns about the quality of imported aggregates. For instance, the quality of imported gabbro has shown considerable variation between and within sources, with specific gravity reported to range from 2.7 to 3.2. This variability causes major problems with the asphalt and concrete mixtures, and therefore Qatar Standards implemented a new system in 2013 for testing the aggregate on arrival in the port before it is allowed into Qatar.

There is a great uncertainty in predicting the quantities of future aggregate demand in Qatar, mainly due to the lack of sufficient information on number of project awarded, start and completion dates and annual aggregate consumption. The total demand for limestone and gabbro aggregates for the period 2012-2022 has been projected to reach 515 and 264 million tonnes, respectively (Kanady, 2013b). Based on these numbers and taking into account the annual consumption until 2014, aggregate demand from 2015 and beyond would increase to more than 80 million tonnes per year, which is much greater than the amounts formerly imported and considerably higher than the current port capacity of 50 million tonnes per year. In addition to this, importing aggregates in such quantities also cause significant air pollution. The CO_2 footprint of transporting quality aggregates to Qatar is roughly 35kg per ton of material (Kemp, 2015) and raises major environmental concerns. Therefore, the constant and sustainable supply of suitable aggregates for the construction industry in the quantities and timescale required is becoming a real challenge.

Similarly, demand for construction sands is on the rise and in 2015 exceeded the 12 million tonnes per year (Al-Hammadi, 2015). Unlike coarse aggregates, the booming construction industry in Qatar is only allowed to utilise the locally mined natural sands which are limited in quantity (reserves are estimated to be about 90 million tonnes) and occur exclusively in the south of the country, as most parts of the country are covered by karstified carbonates of the

Mid Eocene Damman formation, not suitable to be used as construction materials (QPMC, 2013). The only two different types of construction sands available in Qatar are fluvial and aeolian dune sand. Fluvial sand is quarried from several sand pits and processed further (e.g. sieved, graded, washed, etc.) in sand treatment plants to meet the properties set in national construction standards. Fluvial sand is marked as "washed sand" and sold on the market as certified construction sand for concrete works. Conversely, Qatar aeolian dune sand is extracted as bulk product from naturally occurring layers of dunes and with no further processing is sold as "Nijian" sand. Bulk dune sand is usually used for plastering purposes in construction, mortar production and specialised industries applications. (Al-Ansary, et al., 2012).

2.2.3 Recycled aggregates available in Qatar

As part of the development programme in Qatar, old building and structures are being demolished and replaced with new construction throughout the country, particularly in Doha and the nearby cities. This generates huge amount of construction and demolition debris each year, mainly compromised by concrete (used or returned), asphalt and brick debris, and limestone wastes from reservoir excavations and cut-and-fill road works (excavation wastes). Construction and demolition wastes constitutes the main source of solid waste generation in Qatar and usually sent to landfills without undergoes any recycling process, posing risks to the local and regional environment (Clarke and Almannai, 2014).

Indeed, according to the statistics available for the period of 2008 to 2012, reported by the Ministry of Development Planning and Statistics in 2013, Qatar generates around 10 to 12 million tonnes of solid waste every year – almost 75% of which comes from construction and demolition activities (Table 2.1), while recycling rates of solid wastes for the same period do not exceed the 4%. This places Qatar among the countries with the highest per capita waste generation rates worldwide, i.e. 1.8 kg/capita/day (Zafar, 2014), when the rate for most European countries is below 1.3 kg/capita/day (Eurostat, 2014). Existing stockpiles of unprocessed wastes are estimated to be in the order of 60 to 80 Mt, and with no implementation of recycling policies the levels of C&DW are set to double as the state steps up its building programme as part of the Qatar National Vision 2030 infrastructure development plan and ahead of hosting the 2022 FIFA World Cup.

Table 2.1: Amount of incoming waste to landfills by type and landfill in 2010 (Ministry of Development Planning and Statistics, 2013)

Landfill sites	Type of waste, tonnes					Total, tonnes
	Domestic	Construction	Bulky	Tire	Other	
Umm Al Afai	846,630	338,987	1,748,989	601,847	5,030	3,541,483
Rawdat Rashid	-	8,864,475	-	-	-	8,864,475
Total, tonnes	846,630	9,203,462	1,748,989	601,847	5,030	12,405,958
As percentage of total wastes	6.5	74.2	14.1	4.9	0	100

The Qatar Government having identified the need to mitigate the waste's environmental impact have made plans that addresses in Qatar National Development Strategy 2011-2016 (QGDP, 2011). According to this strategy, the Government prioritizes actions to reduce the pressure on the environment, with the most preferable goal being the avoidance of waste generated by households, commercial sites and industry, and where waste cannot be avoided, the preferred goals would be to reduce it, reuse it and recycle it, before wastes disposed into landfills. The desirable recycling rate of the country's total solid waste has been set up to 38% by 2030, up from the current figure of 4%. Waste management, therefore, became a major concern for the government of Qatar as the handling of construction and demolition wastes turns out to be a real environmental challenge.

In countries like the state of Qatar, where aggregate and sand recourses are limited, construction and demolition debris can be a valuable resource of aggregates, except from the obvious environmental and economic benefits that recycling adds, i.e. conservation of natural resources, saving of energy in production and transportation and reduction of pollution. Many initiatives have been undertaken recently by the government and implemented by various ministries and departments with the support of companies and other organisations towards more sustainable construction solutions.

One of these was a collaboration between TRL and Qatar Standards (Ministry of Environment) supported by Qatar University and Qatar Construction & Development Company (a local aggregate supplier), looking into the use of both fine and coarse recycled and secondary aggregates in high-value applications. Preliminary laboratory results and in situ trials have

demonstrated that coarse recycled and secondary aggregates can successfully replace NA in unbound pavement applications and in structural concrete, non-structural concrete and concrete blocks. On the contrary, fine recycled aggregates have been found unsuitable for use in concrete as they display a high fines content and significant amounts of chlorides and sulphates. An alternative was the use of gabbro or limestone crushed rock fines (CRF) as a partial replacement for washed sand, where up to 60% replacement levels were noted to have no detrimental effect on concrete strength (Hassan, et al., 2014).

Consequently, the latest edition of the Qatar construction specifications, published in 2014, permits the use of recycled aggregates from excavation wastes (EW) and CCA as coarse aggregate in concrete at up to 20% replacement of imported gabbro or limestone for structural concrete and up to 50% for non-structural concrete. Recycled aggregates are also permitted up to 100% in sub-base applications. Similarly, excavated limestone waste is permitted as aggregate in asphalt in the lower layers of the pavement, but not in the surface course. Provision for the use of fine recycled aggregates has not been introduced yet.

In this regard, a recycled aggregate plant was officially launched in 2015 at Rawdat Rashid site, the biggest landfill site in Qatar. The plant delivers around 2 million tons of recycled aggregates and aims to increase recycled aggregate production to 15 million tons until 2020 (Kemp, 2015). The use of recycled aggregates is expected to yield major economic and environmental advantages when compared to the use of imported aggregates. A comparison of the likely costs of local recycled aggregates in Qatar indicated a significant reduction of 60% compared to imported aggregates. In addition to this, CO_2 savings of almost 50% can be achieved by the use of local recycled aggregate due to the energy saving in transport that can be avoided in shipping the primary aggregates to Qatar (Kemp, 2015). Limitations on port capacity and concerns about the quality of imported aggregate would also support greater uptake of the recycled materials in construction (Hassan, 2015).

2.2.4 Conclusions

Although, there is a considerable body of national and international research related to using recycled aggregates in the production of concrete products, Qatar just recently has introduced in its construction standards specifications for the incorporation of these materials and as a result the industrial implementation of recycled aggregates is still in its infancy. Maximising the use of recycled aggregates is a key priority for Qatar to meet the demand for constructions

aggregates in the years leading up to the 2022 World Cup, conserve natural recourses and achieve sustainability in the construction industry.

2.3 **Coarse Recycled Aggregates**

2.3.1 Definition

Recycled aggregate is the generic term for aggregate resulting from the re-processing of inorganic material previously used in construction, e.g. crushed concrete, brick and masonry (BS 8500-1, 2015). In addition to significant quantities of natural aggregates and crushed concrete, recycled aggregates can also contain substantial quantities of impurities such as metal, asphalt, wood, glass and plastic; these need to be controlled to acceptable levels dependant on their proposed use. Recycled aggregates (RA) resulting from crushed concrete and containing small percentages of masonry bituminous or other materials (10% in total) constitute a special class of recycled aggregate within BS 8500-1: 2015 named crushed concrete aggregates (CCA, previously known as RCA), while the RA is a less demanding class and may be composed entirely of masonry. The performance characteristics of CCA are regarded as better than those of RA and consequently there are fewer restrictions on their use in concrete. This literature review will concentrate on the characteristics of, and performance of concrete made with, crushed concrete aggregates.

CCA is not the same as "recovered" concrete aggregates, which these aggregates are obtained by washing the cement paste out of fresh concrete and returning the aggregate to the concrete producer. However, it does include aggregates obtained from surplus concrete (i.e. fresh concrete that returned to the concrete plant and allowed to harden before crushing).

2.3.2 Manufacture and processing

By far the largest quantities of recyclable materials suitable for use in new construction come from the broad field of construction and demolition waste (C&DW) (Pacheco-Torgal, et al., 2013). The highest portion of demolition debris arising is concrete capable of being processed into CCA (Figure 2.1).

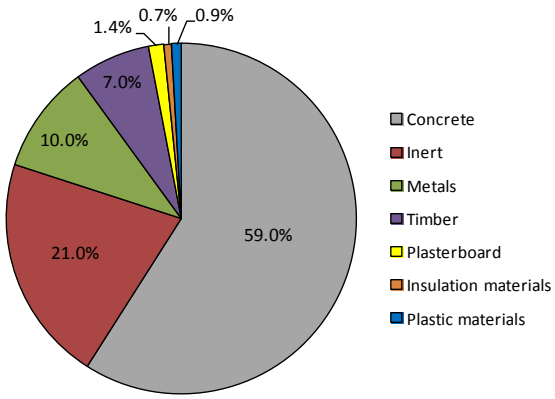


Figure 2.1: Construction and demolition waste composition in Europe (Pacheco-Torgal, et al., 2013)

Recycling of C&DW can take place either at the site from which the material is sourced using mobile crushers, or the material may be transported to a recycling plant, where large stockpiles may be accumulated, for further processing. Mobile crushers are often more economical in that they avoid transporting C&DW away from site, although are rarely sophisticated enough to remove all impurities. For that reason, recycled aggregates produced from mobile crushers are usually used as site fill or sub-base road materials, and used at, or close to, the location they were crushed. To obtain recycled aggregates with properties suitable for use in higher value construction applications, operations should be put in place to restrict the type of C&DW that is used, separate material before crushing, crush twice using primary and secondary crushers, separate impurities and wash the aggregates.

C&DW recycling plants consist of complete processing systems for separating all fractions of mixed C&DW, crushing, screening and removing contaminants. Particularly, protruding pieces of reinforcing steel, metals, wood, glass, tiles, plastics, gypsum, rubber, etc are first removed by hydraulic means or hand picking. Then the material to be recycled is fed into vibrating feeders, which guarantee a uniform and continuous flow into a single or several crushing units. Most recycling plants use a jaw crusher for primary crushing because it can handle large pieces of concrete and residual reinforcement. Jaw crushers also reduces material down to 60-80 mm. Ferrous metal separation follows using large electro-magnets placed over the C&DW flux (cross belt) before the material is fed into a secondary crusher. Impact crushers are preferred for secondary crushing as they produce a higher percentage of aggregate without adhered mortar. In general, the shape of recycled aggregate is rounder and less flaky than natural aggregate. Next the material from the secondary crushing passes through two

vibrating screens that separate the aggregate into sizes greater than 20 mm and between 4 mm and 20 mm, with the material finer than 4 mm being removed. The plus 20 mm material is fed back into the secondary crusher. The 4-20 mm fraction is screened to produce coarse aggregate complying with the grading requirements of the respective standard. Throughout this process, air sifters, non ferrous metal separation, air jigs and water treatment are usually used to improve the quality of the recycled aggregates i.e. less contaminants, dust or organic matter. Figure 2.2 illustrates the typical steps required to process recycled aggregates.

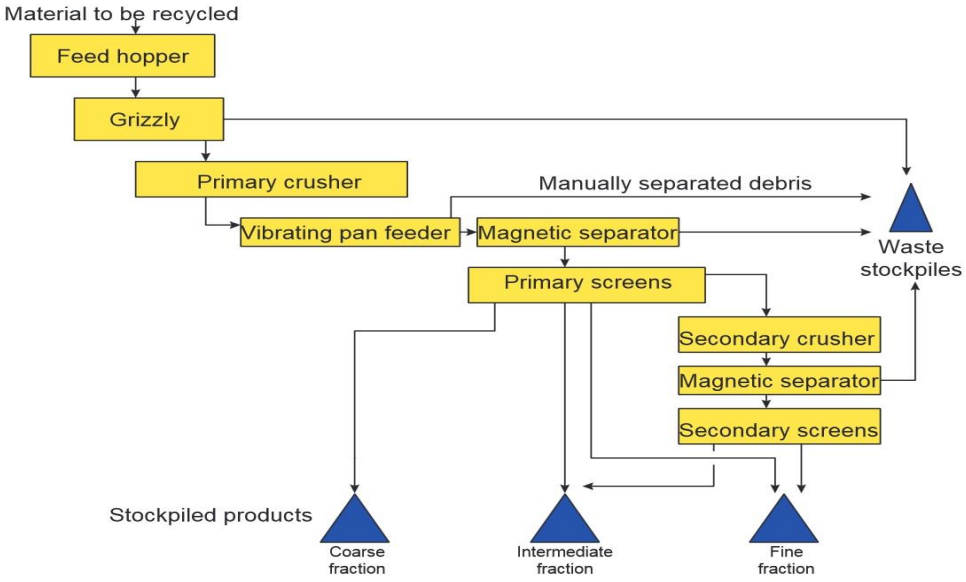


Figure 2.2: Generalized flow diagram for an aggregates recycling operation (Wilburn and Goonan, 1998)

2.3.3 Specifications/Standards covering the use of recycled aggregates in concrete

The current British/European harmonised product standard, BS EN 12620 (BSI, 2002) for *Aggregates for concrete* makes no distinction, in terms of properties covered, between natural, recycled or manufactured aggregates and mixtures of these materials. For example, the recycled aggregates should conform to the same grading requirements as natural aggregates. The British Standards for concrete, BS 8500-2 (BSI, 2015) and BS EN 206 (BSI, 2013), apply then product-specific and specification-specific conditions on the use of coarse recycled aggregates in concrete and gives compositional limits for RA and CCA.

Table 2.2 outlines the limits/recommendations for using coarse recycled aggregates in concrete according to the aforementioned standards. The limits specified by Qatar Construction standard, QCS 5-2 (QS, 2014), are also included in Table 2.2 for comparison. As it can be seen, stricter limits are applied to both CCA and RA.

Table 2.2: Recommendations for coarse recycled aggregates according to Bristish and Qatar standards

Property	BS EN 12620 (2002)	BS EN 206 (2013)	BS EN 8500-2 (2015)	QCS 5-2 (2014)
Fines content (≤63μm)	≤ 4% w/w	Value to be declared	≤ 4% w/w	CCA: ≤ 5% w/w RA: ≤ 3% w/w
Flakiness Index	≤ FL_{50} (category)	≤ FL_{50} (category)	_ [A)]	≤ FL_{35} (category)
LA coefficient	≤ LA_{50} (category)	≤ LA_{50} (category)	≤ LA_{40} (category)	≤ LA_{30} (category)
Oven-dried particle density	≥1 500 kg/m³	Type A: ≥ 2100 kg/m³ Type B: ≥ 1700 kg/m³	_ [A)]	≥ 2000 kg/m³
Water absorption	Value to be declared	Value to be declared	_ [A)]	≤ 3% w/w (structural) ≤ 4% w/w (non-structural)
Constituents category	R_{c50}, R_{cu50}, R_{a10-},R_{b50-}, FL_{5-}, XR_{g2-}	Type A: R_{c90}, R_{cu95}, R_{b10-}, R_{a1-}, FL_{2-}, XR_{g1-} Type B: R_{c50}, R_{cu70}, R_{b30-}, R_{a5-}, FL_{2-}, XR_{g2-}	CCA: R_{c90}, R_{cu90}, R_{b10-}, R_{a5-}, FL_{2-}, XR_{g1-} RA: R_{cNR}, R_{cuNR}, R_{bNR-}, R_{a10-}, FL_{2-}, XR_{g1-}	CCA: R_{c95}, R_{cu95}, R_{b5-}, R_{a5-}, $FL_{0.5-}$, XR_{g1-} RA: R_{cNR}, R_{cuNR}, R_{b10-}, R_{a10-}, FL_{1-}, XR_{g1-}
Water soluble sulfate content	≤ 0.2% w/w	≤ 0.7% w/w	_ [A)]	NR
Acid-soluble sulfate content	≤ 0.8% w/w	_ [B)]	CCA: ≤ 0.8% w/w RA: _ [B)]	CCA: ≤ 1.0% w/w RA: _ [B)]
Acid-soluble chloride content	Value to be declared	Value to be declared	_ [A)]	≤ 0.03% w/w
Influence on setting time	≤ 40mins	≤40mins	_ [A)]	NR
Drying shrinkage	≤ 0.075%	≤ 0.075%	_ [A)]	≤ 0.075%

[A)] : Refer to BS EN 12620 and/or BS EN 206

[B)] : The appropriate limit needs to be determined on a case-by-case basis

NR : No requirement

Coarse RA and CCA conforming to the requirements of BS 8500-2 (BSI, 2015) can be used in both Designated and Designed concrete. In Designated concretes C20/25 to C40/50, a maximum of 20% of the natural coarse aggregate can automatically be replaced by RA or CCA. The use of RA or CCA in FND or PAV types (Table A.14, BS 8500-2, BSI, 2015) of designated concretes is prohibited. For GEN0 to GEN3 designated concretes and designed concrete there are no general restrictions on the aggregate replacement proportion, as long as any durability criteria are met. In addition, coarse RA and CCA can be used in concretes of strength classes up to C40/50 and in most exposure conditions, except to that referred to exposure to salt (XS and XD), severe freeze-thaw (XF2-XF4) and aggressive ground more severe than DC-1. Use in that exposure classes is possible if it is demonstrated that the resulting concrete is suitable for the indented purpose.

The use of RA in concrete is not so widely accepted due to its wide range of composition, and therefore it is not possible to permit its use for a given type of concrete without the need for additional provisions in the specification, based on the composition of the proposed RA (BS 8500-2, 2015). The new version of BS EN 206 (BSI, 2013) indeed specified wider RA applications in concrete with the RA classification based on its constituents (Table 2.3).

Table 2.3: Maximum percentage of replacement of coarse recycled aggregates according to BS EN 206 (BSI, 2013), % by mass

Recycled Aggregate Type	Exposure Classes			
	X0	XC1, XC2	XC3, XC4, XF1, XA1, XD1	All other exposure classes*
Type A: R_{c90}, R_{cu90}, R_{b10-}, FL_{2-}, XR_{g1-}	50%	30%	30%	0%
Type B$^{\$}$: R_{c50}, R_{cu70}, R_{b30-}, FL_{2-}, XR_{g2-}	50%	20%	0%	0%

* Type A recycled aggregates from a known source may be used in exposure classes to which the original concrete was designed with a maximum % of 30%

$ Type B recycled aggregates should not be used in concrete with compressive strength classes >C30/37

Similarly to British Standards, QCS 5-2 (QS, 2014) allows for structural concrete applications CCA to be used in Designated concretes of maximum C30. Its proportion shall be not more than a mass fraction of 20% of coarse natural aggregates. For non-structural concrete applications, such as crash barriers and soakaways, CCA can be used in Designated concretes of maximum C40 at a maximum mass fraction of 50% of coarse natural aggregates. RA can

only be used for non-structural Designated concretes of maximum C40 and at a maximum replacement level of 20%.

Provisions for the use of fine CCA and fine RA are not given neither in BS 8500-1 (BSI, 2015) nor in QCS 5-2 (QS, 2014), however this does not preclude their use where it is demonstrated that significant quantities of deleterious materials are not present and their suitability for a certain concrete application has been proved.

2.3.4 Characteristics of RA/CCA

When compared with the typical natural aggregates used in concrete, cement paste has lower density, higher water absorption capacity, lower Los Angeles abrasion resistant coefficient and higher sulphate/chloride content (Behera, et al., 2014). It is these differences in cementitious mortar and natural aggregate properties that have been recognised as the main factors contributing to quality issues relating to CCA. As the mortar content varies widely from one CCA sample to another, the negative effects of the presence of mortar on a particular property of CCA are proportional to its content. In addition, the relatively severity of impact of such effects is basically a function of the ratio between values of the respective property for mortar and natural aggregates. In the following sections the most important properties determined regularly to assess the quality of CCA are described and the relationships between these properties and mortar content are discussed.

Constituents/Contaminants

According to BS EN 933-11 (BSI, 2009), constituents of the coarse RAs are described as in Table 2.4. Recycled aggregate consists predominately of crushed concrete (R_c) and unbound stone (R_u), and to a lesser proportion of crushed brick and tiles (R_b), although small percentages of asphalt (R_a), glass (R_g), and other constituents (X), such as soil, metals, gypsum, rubber, plastic, or wood may also be present. R_a, R_g and X are usually regarded as contaminants which can severely affect the mechanical performance of concrete made with them and therefore specifications place strict limits on these.

Glass (R_g) is usually removed from buildings prior to demolition activities, and given recycling efforts in most of the European countries tends not to be present in C&DW. This pre-sorting is of vital importance since its density is similar to that of stone and brick, which makes it difficult to separate glass from the rest heavy-weight materials through air sifting and wet separation

procedures. It usually ends up to fines fraction following the crushing procedures in the recycling plants because it is a brittle material. BS EN 12620 (BSI, 2002), as well as many other specifications for recycled aggregates, specifies the glass content not to exceed 1% by mass in CCA and RA.

Bituminous materials (R_a) have been reported to generally reduce concrete strength in a similar way to that of low-strength lightweight aggregate on conventional concrete (Sagoe and Brown, 1998). Hansen (1992) noticed that the addition of 30% by volume of asphalt to recycled aggregate concrete reduced the compressive strength by about 30%. Other authors (Huang, et al, 2006) reported a compressive strength loss of 75% with a replacement level of 64% by weight of total aggregate content. BS EN 12620 (BSI, 2002) allows the use of RA containing a maximum bituminous materials content of 10%. However, BS 8500-2 (BSI, 2015) limit the maximum bituminous content to 5% for CCA.

Table 2.4: Constituents of coarse recycled aggregates (BS EN 933-11, BSI, 2009)

Constituent	Unit	Description
R_c	%	Concrete, concrete products, mortar Concrete masonry units
R_u	%	Unbound aggregate, natural stone Hydraulically bound aggregate
R_b	%	Clay masonry units (i.e. bricks and tiles) Calcium silicate masonry units Aerated non-floating concrete
R_a	%	Bituminous materials
R_g	%	Glass
X	%	Other: Cohesive (i.e. clay and soil) Miscellaneous: metals (ferrous and non-ferrous), non-floating wood, plastic and rubber Gypsum plaster
FL	cm^3/kg	Floating particles (particles that float in water)

In other constituents (X), any non-floating wood, plastic and rubber remaining within RA are often difficult to separate from C&DW prior to crushing and therefore are usually removed using air blowers, water processing techniques or sometimes by hand as the material is

moving upon a conveyor belt between the primary and secondary crusher. Soil and clay particles frequently stick to the aggregate surfaces and if not removed, their presence may cause workability, shrinkage or scaling problems to concrete. A good practice is to wash the recycled aggregates prior to use. Ferrous metals are usually removed from C&DW by means of magnetic belts after primary crushing, whilst eddy currents may be used to remove non-ferrous metals. Gypsum is also classified in the family of "other constituents", however, it is the main source of sulfate content, which is also limited in the chemical composition part of the standard. BS EN 12620 (BSI, 2002) limits the "other constituents" mass to a maximum of 1% by mass of aggregates.

Wood and plastic, which float in water, are classified separately as floating non-stone material and content is limited to a maximum of 2 cm^3/kg of aggregates.

Physical properties

Physical properties of RA considered cover mainly its morphology, density, water absorption, and porosity, which are thought to have significant effects on concrete properties made with them. The morphology refers to the shape and size of the RA particles. Aggregate shape and size appear to be some of the key parameters characterizing the quality of the aggregate, since they do influence the properties of concrete. Researchers (Bouquety, et al., 2007; Chidiroglou, et al., 2008; Fonseca, et al., 2011) noticed that when the portion of flat particles in concrete increases, the workability decreases, as well as the packing density and the compression strength. Similarly, coarseness and increased angularity of the resulting aggregates are the reasons why the workability of concrete made with them may sometimes be a problem. In the majority of the studies on recycled aggregates has been reported that CCA tend to have very similar particle size distribution curves (within the limits sets by national/international specifications) and rounder shape with smoother surface texture than the aggregates derived from crushed rock (Lin, et al., 2004; Etxeberria, et al., 2007). Etxeberria et al. (2007) highlighted that the method of producing CCA, in terms of the type of crushers (e.g. jaw, impact, cone, hammer crushers, etc.) used in the process to break down large pieces and the number of processing stages (primary and secondary crushing), is influential in the shape and size of CCA produced. Jaw crushers, which are usually used in the primary crushing stage, provide the best particle size distribution of RAs for concrete; however the resulting aggregates tend to be flat and sharp (Nagataki, et al., 2004; Etxeberria, et al., 2007; Fonseca, et al., 2011). For that

reason the aggregates usually undergo a secondary crushing stage, where cone or impact crushers are usually used as they produce aggregates with a good size distribution, a more spherical shape and lower flakiness index. This allows the fresh mortar to flow better around the aggregate.

Density is a fundamental parameter of aggregate and is important to concrete mix design and control of several properties of the resulting concrete. Generally, coarse CCA belong to normal weight aggregates; however their densities (range approximately from 2300 to 2500 kg/m^3) are lower than that of natural aggregates (usually > 2500 kg/m^3). This is mainly due to the existence of porous and less dense cement paste in the CCAs (Hansen, 1992; Barra and Vázquez, 1998; Katz, 2003; Juan and Gutiérrez, 2009; Behera et al., 2014). The mortar content in CCA samples reported by Akbarnezhad et al. (2013b) to range from 20 to 70% (by mass), depending on the crushing procedure and the number of processing stages used during production, the strength of mortar in the original concrete and the particle size (Akbarnezhad et al., 2013a). As a result of this variation in mortar content, CCA may have different densities as well. As cement paste is less dense than natural aggregates, the more adhered mortar in CCA the lower the density of the aggregates (Figure 2.3). The amount of mortar adhered to the surface of aggregates is determined by the number of processing stages. The more stages there are, the higher the density of the resulting aggregates due to the cumulative breaking up of adhered cement paste on the surface of the coarse CCA (Nagataki, et al., 2004); however, the CCA will be too fine to be used as coarse aggregates in concrete if too many processing stages are applied.

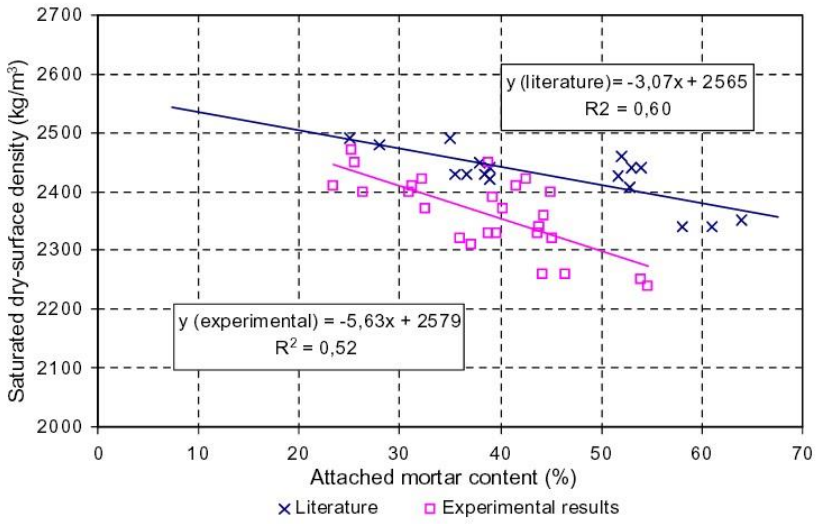


Figure 2.3: Relationship between mortar content and saturated surface-dry density (Juan and Gutiérrez, 2009)

Naturally, the type of the original material has some effect on the density of the resulting aggregate. The densities of bituminous and ceramic materials, as well as other impurities such as gypsum, are lower than that of CCA because of their higher porosity levels, and hence can result in a lower density. Although in CCA the content of these constituents is limited, in RA a decrease is expected as the impurity content is increased (Bazaz, et al., 2006; Khalaf, 2006).

The strength of the parent concrete also has some effect on the density of the CCAs. Concrete mixes with enhanced strength usually require greater cement contents, which, besides increasing packing and yielding a more resistant cement paste, results in a less porous concrete, and so it could be expected that this would increase the density of the resulting aggregates. Gokce et al. (2011) reported that the saturated-surface dry (SSD) densities of CCA derived from three concrete mixes with strength of 60.7, 49.0 and 28.3 MPa were 2420, 2410 and 2370 kg/m^3, respectively. In some other studies (Dhir, et al., 1999; Chandra, 2004) have been established that CCA from concrete materials of varying compressive strength and subjected to similar recycling procedures had essentially similar densities. However, Poon et al. (2004a, b) reported lower densities for aggregates obtained from high strength concrete (HPC) than that of recycled aggregates made from normal strength concrete.

A clear relationship between the size of recycled aggregates and their density was found in many studies (Poon and Chan, 2006; Juan and Gutiérrez, 2009; Tam and Tam, 2009). Particularly, in one of these studies (Juan and Gutiérrez, 2009), the influence of the attached mortar content on the properties of CCA was investigated and was found that the attached cement paste content increases as the fraction size decreases. This can be explained by the recycling processes and the use of several mechanical processing stages as mentioned above. Considering this, it is expected that the density of fine recycled aggregate is lower than that of the coarse fraction, from the same origin.

When water added to the mix, aggregates tend to absorb part of the mixing water depending on their porosity and therefore the role of aggregates' water absorption becomes a decisive matter during mixing of concrete, as it affects the effective w/c. Recycled aggregates will almost always exhibit higher water absorption values than natural aggregates, in the range of 3% to 10% by mass (Etxeberria, et al., 2007), the extent of which vary according to the contents of materials such as unbound and bound aggregates, concrete, brick, tiles, asphalt and other contaminants. It is well studied that these contaminants are porous by nature and absorbs high

amounts of water (Poon and Chan, 2006). In case of CCA, where most of the contaminants have been removed, the water absorption rate is increased by the presence of adhered mortar (Figure 2.4). The quality of the parent content, in terms of porosity of cement paste, was also found to have a slight influence on the water absorption of the resulting aggregates. According to Andreu and Miren (2014), recycled concrete aggregate obtained from concretes with lower w/c ratio, and so with lower porosity and enhanced mechanical strength, exhibit lower water absorption. In another study (Katz, 2004) however, the water absorption found to remain pretty well unaffected as the strength of the original concrete increased.

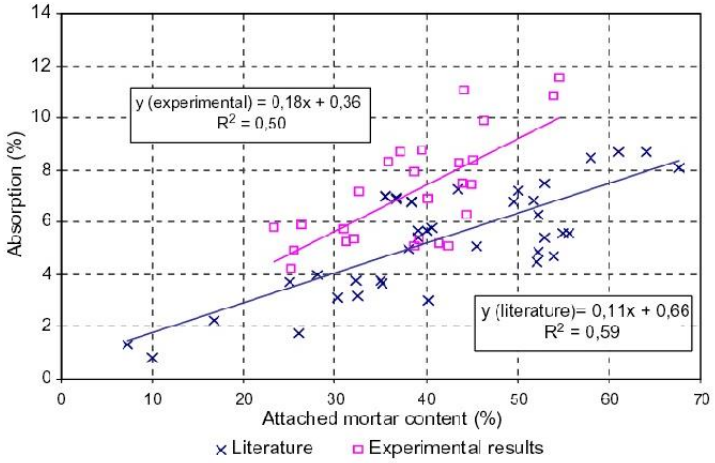


Figure 2.4: Relationship between attached mortar content and water absorption capacity of recycled concrete aggregate (Juan and Gutiérrez, 2009)

Coarse recycled aggregates tend to have lower water absorption than the corresponding fine fraction (Chandra, 2004; Katza, 2004; Poon and Chan, 2006). This can be explained by the relatively higher amounts of crushed cement paste and fine particles accumulating in the fine aggregate fraction after various processing stages. Washing crushed fine or coarse recycled concrete aggregates is well known process of reducing the resulting water absorption, since the very fine particles, which conferred quite water absorptive, are removed (Van Der Wegen and Haverkort, 1998).

Investigation on water absorption capacity of coarse CCA as a function of time (Ferreira, et al., 2011; Vieira, et al., 2011) indicated that the water absorption of the CCA occurs mainly during the initial immersion period, reaching about 80-90% of its absorption potential after only five minutes of immersion. After this period, it is increasing in a much slower rate. If CCAs are not pre-treated, this fast and higher water absorption capacity will imply a lower degree of workability for the same w/c ratio of concrete containing CCA and then more mix water is

required to mix the cement paste. Since avoiding water absorption problems by waterproofing the recycled aggregates is impractical and expensive, researchers (Barra and Vazquez, 1996; Poon et al., 2004a) proposed the two following methods to overcome the loss of workability and reduction of effective w/c ratio when recycled aggregates are used: i) extra water is added to the mix, corresponding to the water absorbed by the recycled aggregates (mixing water compensation method) and ii) recycled aggregates are added to the mix saturated, cancelling out water absorption from the mix or nearly saturated, i.e. in saturated and surface dry condition (SSD), making water absorption insignificant (pre-saturation method).

The mixing water compensation method has the advantage of making it possible to produce both conventional and recycled aggregate concrete in a similar way. The amount of extra water added on the mix depends on the initial moisture content and effective absorption of recycled aggregates during the mixing period. Potential water absorption and absorption evolution with time should also be known in order to predict the w/c ratio after the mixing period. This ought to guarantee that the additional water does indeed correspond to the amount of water absorbed by the aggregates. However, Oliveira and Vazquez (1996) underlined that the water absorption of recycled aggregates may not correspond to the absorption of free water determined in the laboratory, since their pores are filled with cement paste during mixing, which may lead to an excess of free water in the mix, and hence to an undesirable increase in the effective w/c ratio. This method has an additional problem related to the variation in the water absorption capacity of the recycled aggregates, which means that the extra water is not a fixed value since recycled aggregates presenting an inherent heterogeneity.

On the other hand, pre-saturation method aims to prevent water exchange between the aggregates and the cement paste. The main issues arising from that method are to define the quantity of water needed for the pre-saturation and how to achieve it. Oliveira and Vazquez (1996) and and Poon et al. (2004a,b) propose the pre-saturation of coarse recycled aggregates in a SSD condition before mixing, and not in a fully saturated condition as suggested by Nealen and Schenk (1998), to avoid any 'bleeding' effect, that is the transfer of water from within the aggregates to the cement paste. Such 'bleeding' could alter the w/c ratio in the interfacial transition zone (ITZ) between the aggregates and the cement paste and affect the bond strength (Poon, et al., 2004b). Tests on several concretes with different incorporation ratios of recycled coarse concrete aggregates and in different moisture conditions (i.e. oven dried, air dried and

saturated surface dried) showed that the mechanical performance of concrete prepared by pre-saturated followed by 30 min air-dried CCA (semi-saturated) was better than the performances of concrete containing oven dry or saturated and surface dry CCA (Oliveira and Vazquez, 1996; Poon, et al., 2004a). The humidity level of air-dried CCA was about 90 % with respect to saturated surface dry aggregate. A humidity level of 80% of the total absorption capacity of recycled aggregates was also recommended by Etxebrria et al. (2007).

Porosity of CCA is closely linked to density and absorption. As these properties are highly affected by the content of attached mortar, porosity of CCA rises significantly when the amount of cement paste is increased and aggregate size is smaller (Etxebrria, et al., 2007; Tam and Tam, 2009; Juan and Gutiérrez, 2009).

Poon et al. (2004a), who evaluated the mercury intrusion porosity of natural coarse aggregate (NA), CCA prepared from normal concrete (NC) and CCA prepared from high-performance concrete (HPC), found that the porosities of NA, NC-CCA and HPC-CCA were 1.60, 16.81 and 7.86 %, respectively. According to the authors, the higher porosity of CCA aggregates is attributed to the adhered cement paste, whereas the lower porosity of HPC-CCA was due to the use of pozzolanic admixtures, which substantially improved the microstructure of the cement paste and the paste–aggregate ITZ.

Mechanical performance

The mechanical properties of the parent materials have a significant impact on the mechanical performance of recycled aggregates, mainly defined in terms of its Los Angeles (LA) abrasion coefficient, toughness and soundness. These tests are only performed in the coarse size fraction of the recycled aggregates and the results can be extrapolated to the fine fraction.

Abrasion resistance is an important property of aggregate and is often used as a general measure of the aggregate's quality to resist fragmentation during handling and mixing. The aggregate abrasion value is defined as the percentage loss in weight by abrasion, so that a high value denotes low resistance to abrasion (Neville, 2011). Los Angeles abrasion test is more commonly used all over the world and therefore the abrasion values obtained from this test reported in most specifications. BS EN 12620 (BSI, 2002) specifies a LA coefficient of maximum 50% for aggregates used in concrete production.

CCA are acknowledged to have on average lower abrasion resistance than natural aggregates because of the presence of adhered mortar, which disintegrates easily during abrasion along with some parts of natural aggregates. Nevertheless, this value for the majority of the CCA reported in the literature meets the standard requirements for concrete. Juan and Gutiérrez (2009) found an increasing trend of the Loss Angeles value with increasing adhered mortar content, which is presented in Figure 2.5. It was also reported that LA value of CCA prepared from high strength concrete is higher than that prepared of low strength concrete (Juan and Gutiérrez, 2009; Tabsh and Abdelfatah 2009).

Toughness is the capacity of the aggregate to resist failure by impact. Aggregate toughness is determined using the aggregate impact value, and the lower the aggregate impact value the tougher the aggregate. An acceptable limit of this value for heavy-duty concrete elements is 25%, whereas 30% is specified for sub-base applications and other low-grade concrete applications (Neville, 2011). CCA exhibit generally a higher impact value than that of normal coarse aggregate because of the separation and crushing of adhered mortars in CCA during testing (Rao, et al., 2011).

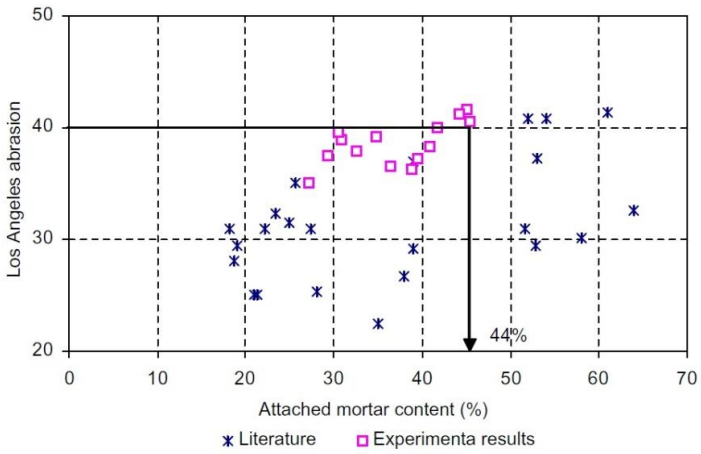


Figure 2.5: Relationship between mortar content and Los Angeles abrasion (Juan and Gutiérrez, 2009)

Soundness determines the mass loss of the aggregate through its resistance to disintegration by weathering and, in particular, freeze-thaw cycles. The durability performance of recycled aggregates is generally inferior to that of normal aggregates and as a result the soundness of CCAs in sulfate solution is poorer than that of NAs because of its marked water absorption. However, some CCA samples meet the standard requirement (Tabsh and Abdelfatah, 2009).

Chemical composition

The history and properties of the parent C&DW materials are not likely be known, either the environments or conditions that these materials have been exposed, and therefore ascertaining the chemical composition (e.g. sulfate, chloride and alkali content) of CCA is of important as that could compromise the performance of concrete.

Soluble sulfur compounds in CCA (sourced mainly from gypsum) are potentially reactive and may give rise to expansion reactions, which significantly reduce concrete durability. Gypsum which occurs in finely dispersed form and originates mainly from plasterwork has a negative effect on material's quality for reasons of solubility, low density and low hardness. Researchers (Barbudo, et al., 2012) found by performing a statistical analysis on recycled aggregates derived from different sources that the soluble sulfate content in both water and acid test is strongly influenced by the percentage of gypsum and crushed clay brick present in recycled aggregate. BS 8500-2 (BSI, 2015) allow a maximum gypsum content in CCA of 1% , however research has shown that up to 5% by mass there was no significant effect on concrete strength or delayed ettringite formation (Dhir, et al., 2001). CCA have generally higher sulfate content than NA because of sulphates from cement of the adhered mortar (Juan and Gutiérrez, 2009). A high correlation was also found between the adhered mortar content and sulfate content (i.e. sample with higher contents of attached mortar had higher sulfate content). BS 8500-2 (BSI, 2015) limits the acid-soluble sulfate content of CCA to a maximum of 0.8% by mass.

Concrete standards place a limit on the chloride content of concrete between 0.2% and 1.0% by mass of cement, based on the sum of the contributions from all constituents. The total chloride content is unlikely to be extracted using water in the procedure described in BS EN 1744-1 (BSI, 2009) even if the sample is ground to a fine powder before extraction (BS EN 12620, BSI, 2002). Chloride contributions from CCA for use in this calculation are measured by an acid-soluble test (BS 1881-124, BSI, 2015) that provides a worst-case value (Dhir, et al., 2001) and probably overestimates the availability of chlorides, thus providing a margin of safety (BS EN 12620, BSI, 2013).

CCA sourced from concrete subjected to marine environments may have a high soluble chloride content (Dhir et al., 2001), and then a specific approach is needed to guarantee a sufficiently low concentration of chloride ions in CCA. Washing the CCA with water is one way of reducing the chloride concentration because they are not linked to the cementitious

microstructure and are thus easy to be removed. After a thorough washing or total immersion in water for at least 2 weeks, the amount of chlorides decreases to a point where these CCA can be used in reinforced concrete without any risk of corrosion (Debieb, et al., 2010).

The presence of alkalis, usually originated from cement, and reactive silica from aggregates may lead to alkali-silica-reaction and the formation of an expansive gel. Concerns are often raised over the use of recycled aggregates in concrete because they can contain fractions of alkali-rich hydrated cement in the crushed concrete fractions, as well as alkalis from the product; however studies (Dhir and Paine, 2004; Dhir and Paine, 2007) has shown that in most cases the total equivalent sodium oxide values for Portland cement concrete containing CCAs were below the recommended limit of 3.5 kg/m^3. An amendment to BS 8500-2 (BSI, 2015) allows the alkali contribution of CCA to be taken as 0.20 kg Na_2O_{eq} per 100 kg of CCA or where the composition of the CCA is known (e.g. surplus precast units) to be calculated from the original concrete, and the aggregates to be treated as having normal reactivity.

Performance-related characterisation

Characterising RA by its constituents and source may not be convenience in practice. A performance-related approach was proposed for use of RA in concrete based on researches carried out in Dundee (Paine and Dhir, 2010). This approach is based on the selection of RA on performance-related characteristics that relate the properties of RA to concrete performance across the whole range of RA quality, independent of constituents and source. In other words, three classes of recycled aggregate can be derived based on the properties (LA, aggregate absorption, density & drying shrinkage) of the combined coarse aggregate. From these, class A may be used in a wide range of concrete, and possibly marine environments, whilst class C should be restricted to only the "mildest" exposure conditions (i.e. XO and DC-1).

In this suggested approach, classification of RA is based on its LA coefficient (BS EN 1097-2, BSI, 2010), density and water absorption (BS EN 1097-6, BSI, 2002), and drying shrinkage (BS EN 1367-4, BSI, 2008) and high quality RA for high-performance applications while the low classes for low performance applications. Given this approach, material that is currently not fully specified for use in BS 8500-1&2 (BSI, 2015) may be classified and considered for relevant applications. This should remove the main technical barrier that is preventing the uptake of RAs in concrete, and lead to greater confidence in specifying and using recycled aggregates. Similar approaches will be used in this project.

2.3.5 Properties of RA/RCA Concrete

The fresh, mechanical and durability properties of concretes containing recycled concrete aggregates have been extensively studied, mainly using the coarse fraction of CCA and to a lesser extent the fine fraction. As a general state, when limited amounts (up to 20%) of coarse CCA are used, the mechanical and durability properties of concrete are hardly affected. When incorporating RA from masonry rubble or asphalt-based materials, the mechanical performance declines (Taha, et al, 2002; Bazaz, et al, 2006) however, these materials showed enhanced performance with the use of RA from crushed concrete, i.e. CCA (Papp, et al., 1998; Lim and Zollinger, 2003; Gabr and Cameron, 2012; Behiry, 2013). This improvement can be explained by the self-cementing properties of the unhydrated cement of the crushed concrete particles and rougher surface, which increases inter-particle friction, causing even load redistributions.

Fresh properties

Consistence of concrete is the property of freshly mixed concrete which determines the ease and homogeneity with which it can be mixed, placed, consolidated and finished without any segregation. It is highly dependent on the water content of the concrete mixture and the properties of its constituents, and control various fresh and hardened-state properties of concrete. For instance, an unworkable concrete may lead to a poor compaction or segregation, trapped air-voids, lower density and subsequently lower strength or high absorption.

When natural aggregates are replaced by CCA, there is a trend towards a reduction in consistence of concrete with an increase in CCA content. This is mainly attributed to higher water absorption capacity of CCA; however, even if the water exchange between the aggregates and the cement paste limited by pre-saturation or other methods, the surface texture and angularity of CCA also have an influence on the consistence of concrete (Buyle-Buddin and Zaharieva, 2002; Poon, et al., 2004b; Yang, et al., 2011). The influence is negligible when only limited amounts of course CCA (up to 30%) are used, while for complete replacement of natural coarse aggregate with pre-saturated CCA the slum loss observed by researchers (Limbachiya, et al., 1998; Yang, et al., 2011) ranged between 15 and 30%. In most cases however, slump remains essentially within specified tolerances (± 25mm) (BS EN 206, BSI, 2013). The effect of CCA on retention of consistence has been also studied with the results

show that the total replacement of natural coarse aggregate by CCA in concrete prolonged the loss of consistence with time (Limbachiya, et al, 1998; Poon, et al., 2004b).

CCAs have lower density than natural aggregates because of the presence of lower density residual cement mortar particles attached to the aggregate. This difference is however not so great, and even when a total replacement of coarse aggregates is used, the final effect on the hardened density of concrete is marginal (less than 5%).

The use of CCA is unlikely to pose any problem for the production of concrete that is cohesive and stable in the fresh state (Dhir et al. 1998). Some researchers have reported concerns with bleeding but this is generally attributable to excessive free water in the mix as a result of over-wetting of CCA or not giving them sufficient time to reach a saturated state in the mix (Limbachiya, et al. 1998; Poon, et al., 2004b).

Engineering properties

The use of coarse CCA in concrete causes a reduction in compressive strength, especially as the substitution level increases. This reported extensively in publications (Limbachiya, et al., 1998; Price, 2002; Katz, 2003; Topçu and Sengel, 2004; Etxeberria, et al., 2007; Kou and Poon, 2008; Yang, et al., 2011) where researchers pointed out a negligible influence (less than 10% decrease) on 28-days compressive strength of low-medium strength concretes (20-45MPa) when the coarse CCA is used to replace up to 30% of coarse NA, and a significant loss of strength (between 20 and 30%) when 100% course CCA are used. Figure 2.6a shows a typical strength development of CCA concrete. In contrast with the majority of the observations, in some studies (Domingo-Cabo, et al., 2010; Rao, et al., 2011) the addition of coarse CCA to concrete does not have adverse effect on its strength performance, possibly due to the reduction in effective w/c ratio on account of the higher water absorption capacity of RCA and their rougher surface texture which helps the bond strength between the aggregate and the cement paste. Figure 2.6b indicates for a given CCA content, concretes produced using CCA obtained from different sources (i.e. original concrete of various strength classes) have no significant variation in strength (Kou and Poon, 2008).

While the quality of CCA does not affect the strength of concrete at normal to high w/c ratios, i.e. for low to medium strength concrete mixes the compressive strength intrinsically depends on the cement paste strength, there is evidence to suggest that the quality of CCA does affect

strength of concrete at low w/c ratios (Ryu, 2002; Poon, et al., 2004a). The use of CCA in high strength concrete requiring a 28-days cube strength greater than 60 MPa is not generally an industry acceptable practice, but if CCA are used, then consideration should be given to using CCA sourced from high strength parent concretes (Limbachiya, et al., 2000, Ryu, 2002; Poon, et al., 2004a).

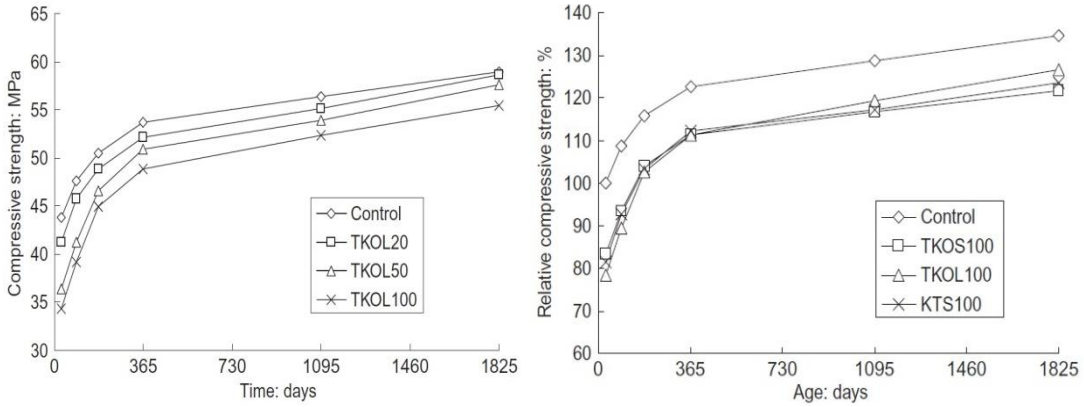


Figure 2.6: a) Development of compressive strength of concrete containing different replacement levels of CCA; and b) Relative compressive strength of concrete with 100% recycled aggregate obtained from three different sources (Kou and Poon, 2008)

The reduction in compressive strength is mainly due to the (i) quality of recycled aggregate reflected in higher porosity, lower density and lower Los Angeles coefficient; (ii) higher water demand for mixing, affecting the w/c ratio of concrete and subsequently its mechanical properties and; (iii) presence of weak zones in the concrete due to the old adhered mortar in CCA (Figure 2.7). Regarding the later, it is believed that the interface between the original aggregate and adhesion mortar (old interfacial zone (ITZ)) and the interface between the adhesion mortar and the new mortar (new ITZ) plays an important role in strength performance of CCA concrete (Ryu, 2002; Otsuki, et al., 2003; Poon, et al., 2004a; Kou and Poon, 2008; Xiao, et al., 2012).

The reduction in compression strength due to the addition of CCA can be controlled by changing various factors of the concrete mix such as adjusting the cement content or w/c ratio, changing the mixing procedure, treating the aggregate and using mineral additions. For instance, Etxeberria et al. (2007) observed that medium strength (30-45MPa) concrete made with 50% or 100% of CCA needs around 4 to 10% lower effective w/c ratio and 5 to 10% more cement than conventional concrete to achieve equal 28-days compression strength.

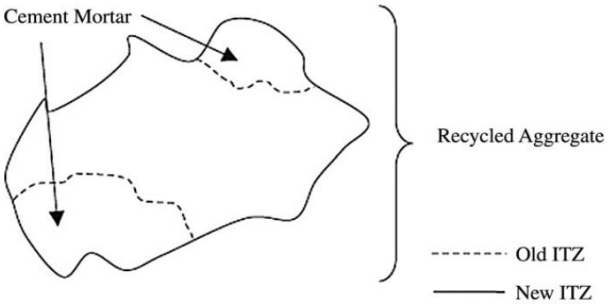
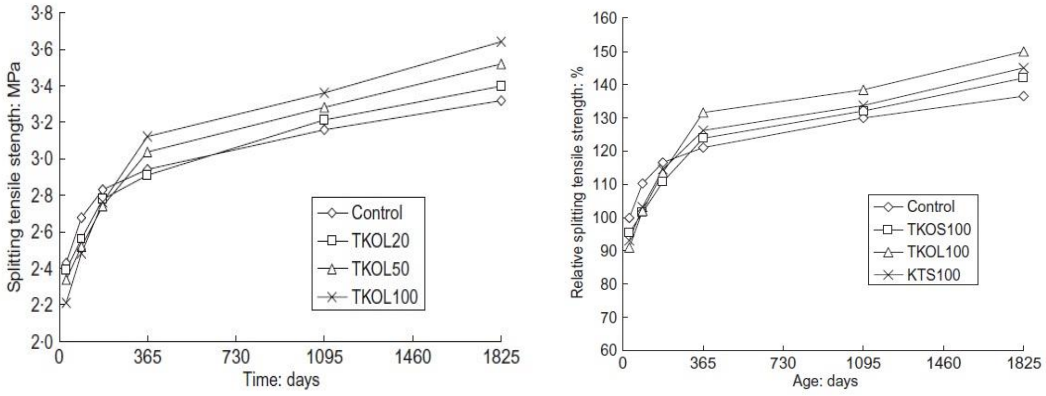


Figure 2.7: Concept of old and new interfacial transition zones (Xiao, et al., 2012)

Like for compression strength, the 28-days tensile splitting and flexural strength of concrete containing CCA is usually unaffected for replacement ratios of natural aggregates by CCA up to 30% by mass, while there is a gradual reduction in these properties compared to conventional concrete as the CCA content is increased (Limbachiya, et al., 1998; Kou and Poon, 2008; De Brito and Robles, 2010; Xiao, et al., 2012). In any case however, the percentage loss was consistently much lower (approximately 10% for 100% CCA) than that observed in compressive strength of CCA concretes. It was also noticed that the CCA concrete performed superior to conventional concrete in a long-term base (after 1 and 5 years of curing) with regards to tensile splitting and flexural (Figure 2.8a,b).

An improvement in the microstructure of the ITZ, increasing the bond strength between the new cement paste and the old aggregates after prolonged hydration, and the self-cementing ability of recycled aggregate were the causes of these observations at the later stages of curing (Kou and Poon, 2008; Kou, et al., 2011; Xiao, et al., 2012).



(a) **(b)**

Figure 2.8: Development of tensile splitting strength of concrete containing different replacement levels of RCA; and b) Relative tensile splitting strength of concrete with 100% recycled aggregate obtained from three different sources (Kou and Poon, 2008)

Shrinkage of concrete with CCA is higher than that of conventional concretes, the increase depending on the replacement level and w/c ratio (Xiao, et al., 2012). Regardless the w/c ratio, it can been seen in various studies (Hansen and Boegh, 1985; Sagoe-Crentsil, 2001; Ajdukiewicz and Kliszczzewicz, 2002; Limbachiya, et al., 2000; Limbachiya, 2010; Domingo-Cabo, et al., 2010) that the limited replacement of coarse natural aggregate (up to 20%) has a negligible effect on shrinkage, while the incorporation of greater amounts of CCA increases significantly the level of shrinkage (reported to exceed 40% for total replacement of NA), possibly due to the lower restraining capacity of CCA particles compared to natural aggregates and the shrinkage of the adhered cement paste (Sagoe-Crentsil, 2001).

Durability performance – Permeability properties

The durability of concrete is defined as the ability of concrete to withstand chemical attack and external environmental and physical actions. It is greatly influenced by its permeability behaviour. The incorporation of RA in concrete can alert concrete's porosity due to their higher porosity compared to NA. Several properties such as water absorption capacity by immersion and capillarity, chloride penetration and carbonation depth are usually evaluated to determine the durability performance of concrete.

Since the water absorption capacity of RA is higher than that of NA, concrete containing RA has higher water absorption capacity than conventional concrete. The water absorption can be evaluated by a capillarity test, e.g. initial surface absorption (ISA), and by an immersion test, which measures the open porosity of concrete specimens.

Dhir et al. (1998) observed that up to 30% CCA, there is no influence on ISA of low-medium strength concrete, and thereafter it increased with CCA content (Figure 2.9). The increase in ISA of CCA concrete was probably due to the increase proportion of cement paste surrounding recycled aggregate, which in case of concrete with 100% CCA the quantity of attached cement paste was approximately 2.5 times greater than in 30% CCA, and acts as a conduit for moisture transport. These results are in line with those observed for high-strength concretes (Limbachiya, et al., 2000). Kwan et al. (2012), using the immersion method, also noticed that replacement of 30% by weight of NA by CCA led to a negligible increase of water absorption capacity, whereas for an 80% replacement, the water absorption capacity was around 2 times higher than that in conventional concrete.

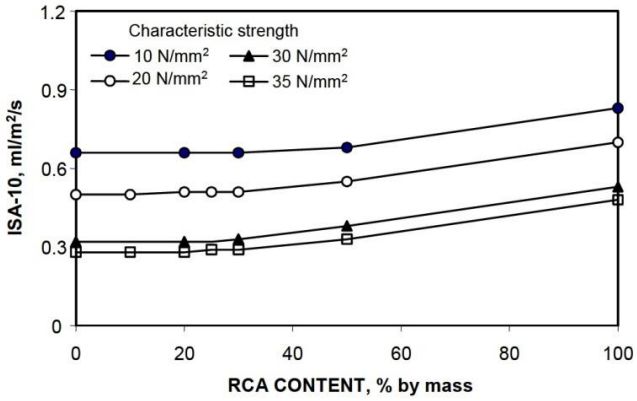


Figure 2.9: Effect of RCA content on initial surface absorption (Dhir, et al., 1998)

The determination of diffusivity of chloride ions through concrete can provide data on the permeability performance of concrete. Another important property that needs to be also evaluated for reinforced concrete structure is chloride-induced corrosion. Several studies were undertaken to understand the chloride permeation and chloride-induced corrosion performance of concrete containing CCA. The incorporation of CCA in concrete generally increases the chloride permeability of concrete (Olorunsogo and Padayachee, 2002; Du et al. 2006; Kou and Poon, 2008; Kou and Poon, 2010); however, different results are also available indicating negligible influence of CCA incorporation on the chloride permeability and chloride-induced corrosion performance of concrete (Limbachiya, et al. 2000).

The evaluation of concrete containing CCA indicated comparable carbonation resistance for concretes with up to 100% CCA content (Dhir, et al., 1998; Shayan and Xu, 2003; Limbachiya, 2010). This performance has been attributed to a number of factors: (i) increase in calcium hydroxide content with more attached cement paste content, and (ii) increase in alkalinity due to increased cement content in CCA concrete to reach equal strength of concrete. However, the carbonation resistance of concrete due to complete replacement of coarse NA by CCA was reported to be inferior to that of conventional concrete by other researches (Sagoe-Crentsil, et al., 2001; Buyle-Bodin and Zaharieva, 2002; Gomes and de Brito, 2009).

Studies (Shayan and Xu, 2003; Limbachiya; 2010, Limbachiya, 2012) focusing on the evaluation of the resistance of CCA concrete to several harmful chemical reactions or chemical environment such as alkali aggregate reaction and sulphate resistance suggest that for replacement ratios up to 30% of coarse NA by CCA there was no noticeable effect on the expansion of the concrete specimens. They also concluded that although CCA concrete may

exhibit higher expansion as the replacement ratio is increased, the expansion is usually between the established limits of specifications and do not cause any damage in concrete. In BS 8500-2 (BSI, 2015) recycled aggregates are classified as high reactivity unless it is made from CCA which only contains normal or low reactivity aggregate.

2.3.6 Conclusions

Coarse recycled concrete aggregate is a very heterogeneous material wherein its quality is highly dependent on the quality of the construction and demolition debris. CCA quality is inferior to those of virgin aggregates; although research has proven that CCA conform to the national/international standards could successfully be used as substitute of natural aggregates to produce normal and high strength concrete, meeting required design strength and desired durability performance. The literature review led to a general conclusion that up to 30% coarse CCA, used as natural aggregate replacement, has no negative effect on fresh and hardened properties of conventional concrete, while higher incorporation levels of CCA affect concrete's properties to a different extent. Currently, BS 8500-2 (BSI, 2015) allows in designated concretes a maximum direct replacement of coarse NA by CCA of 20%; however, in a performance-related approach higher replacement proportions can be used as long as any concrete performance criteria are met. This should lead to greater confidence in specifying and using CCA and thereby help to overcome the main barrier that is preventing the greater use of CCA in higher grade concrete applications.

2.4 **Calcium Sulfoaluminate (CSA) Cement**

2.4.1 Introduction

Many contemporary concrete specifications, in addition to the required engineering properties, aim to reduce the eCO_2 of constituent materials, and most commonly replace part of the Portland cement (PC) content with a pozzolanic additions, typically fly ash or slag. For the vast majority of applications this will continue to be the most cost effective and efficient approach, but in some circumstances, it may be appropriate to utilize alternative non-Portland cements that combine a lower eCO_2 and superior features (e.g. high strength, resistance to sulfates, etc)

The most promising alternative cementing systems for general concrete applications at ambient temperatures currently appear to be those based at least in part on calcium sulfates,

the availability of which is increasing due to the widespread implementation of sulfur dioxide emission controls. These include calcium sulfoaluminate–belite–ferrite cements of the type developed by the China Building Materials Academy in the 1970s under the generic name ''Third Cement Series'' (TCS) and make good use of the potential synergies among calcium sulfate, calcium silicate and calcium aluminate hydrates. These CSA cement binders have the capability of gaining rapidly sufficient strengths and can be used as a replacement to Portland cement to produce rapid hardening high early strength concrete.

The term calcium sulfoaluminate can also be used to refer to Klein's compound. Klein's compound, more commonly known as ye'elimite, was firstly developed in 1960's by Alexander Klein (Klein, et al., 1961) and used as a special cement additive to be blended with Portland cement, mainly to control the dimensional stability of the concrete (achieving shrinkage compensation), as well as providing higher early strength gains (Janotka, et al., 2002; Janotka, et al., 2003). The literature review will focus on CSA cement binders to be used as a cement binder in their own right and particularly to those having belite phase among the major compositional constituents (CSAB).

Despite the fact that CSA cements were developed over 50 years ago and have attracted a lot of interest for their tailored properties, they have only been used on a large scale in China. Widespread use of CSA cements in Europe is strongly limited by the lack of sufficient performance data and standards; current standards consider them as special cements derived from non-Portland clinkers. A substantial body of research does exist but don't consider in depth all the aspects of the material. Most of the research studies examined the hydration and microstructural aspects of CSA-based pastes or in part the mechanical properties of CSA-based concretes. Little attention was given (especially in the western literature) to the long-term durability performance of CSA-based concretes (Ioannou, et al., 2014; Quillin, 2001; Zhang and Glasser, 2005; Glasser and Zhang, 2001, Scrivener and Capmas, 1998), at least towards assessing their applicability for commercial scale use under standardized procedures. Besides that, the wide compositional variation of the material makes it difficult to draw strong conclusions about CSA performance. Subsequently, there is a need to evaluate the true potentiality of its performance prior to introducing them to current design frameworks.

Another drawback for its commercialization is the higher cost compared to PC, which is related directly to the limited supply of bauxite, the main source of alumina in CSA production

(Quillin, 2007). Impure alumina resources are nonetheless fairly widespread and if used can increase the potential for large-scale production of CSA, as is the case in China.

Nowadays CSA cements are receiving renewed interest from cement specialists mostly because its production is considered more sustainable in a number of ways. Firstly, the firing temperature used to produce CSA clinker is typically 1250 °C (Juenger, et al., 2011). As this is about 200°C lower than that involved in PC production, the energy input requirement and its related fuel-derived carbon emissions are lower (Phair, 2006; Gartner, 2004). Besides that, the CO_2 emissions associated with the production of the individual mineralogical cement compounds (Table 2.5), given that a typical CSA consists of predominantly ye'elimite, belite and smaller amounts of aluminoferrite, can be estimated at approximately 600kg/t, which is up to 30% lower than that associated with PC. Additionally, CSA cements can be produced with marginally lower limestone content which its calcination is known to release considerable amounts of carbon emissions. Finally, CSA clinker is also more porous and friable than PC clinker and therefore less energy is required during the grinding stage which further reduces CO_2 emissions (Ludwig and Zhang, 2015).

A further aspect to be considered is the eCO_2 of CSA cement in relation to other cement combinations. For example the calculated eCO_2 value of around 600 kg/t is still higher than the average values of conventional blast-furnace cements (CEM III). Therefore, to strengthen the potential to offer substantial eCO_2 reductions for achieving equivalent concrete performance there is a definite need to develop chemically stable and dense CSA cement combinations that makes maximal use of additions.

Table 2.5: CO_2 released from calcination during the manufacture of "pure" cement compounds (Gartner, 2004)

Phase		t CO_2, t of component
Alite	C_3S	0.578
Belite	β-C_2S	0.511
Tricalcium aluminate	C_3A	0.489
Calcium aluminoferrite	C_4AF	0.362
Monocalcium aluminate	CA	0.279
Ye'elimite	$C_4A_3\hat{S}$	0.216

C = CaO, S = SiO_2, A = Al_2O_3, F = Fe_2O_3, $\hat{S}$ = SO_3, $\bar{C}$ = CO_2 and H = H_2O

2.4.2 Composition of CSA cements

Ye'elimite containing clinkers are typically produced by mixing calium carbonate, perhaps as limestone, aluminous clay and/or bauxite and gypsum, as sources of calcium, silicon/aluminum and sulfur, respectively. For CSAB clinker preparations silica sand and a small amount of ferrolite are also used (these minerals may naturally co-exist with some low quality bauxite mines in China). The availability of waste materials containing high amounts of alumina (such as fly ashes and aluminum anodization muds) or sulfates (such as phosphogypsum and flue gas desulfurized gypsum) is a key issue for the economic production of CSA cements (Pace, et al., 2011; Singh and Pradip, 2008). Even aluminium-poor waste materials, such as municipal solid waste incineration fly ash, have been used as raw materials for CSA clinker preparations (Shi, et al., 2011; Wu et al., 2011).

In terms of mineralogy, CSA clinker is a complex material and the individual composition for a CSA cement can vary quite a lot within literature. The lack of defined or specified phase assemblages is an important fact to be kept in mind as different compositions may have different characteristics and show different behaviour in terms of fresh, engineering and durability properties. Nevertheless, all CSAB cements contain ye'elimite phase (typically in the range of 50-75%), belite (usually below 35%) and small amounts (below 10%) of calcium aluminoferrite (Aranda and De la Torre; 2013; Garcia-Maté, et al., 2012; Zhang and Glasser, 1999). Minor accessory phases such as gehlenite, mayenite, $C_{12}A_7$, CA can also be found in CSA cement clinkers (Glasser and Ahang, 2001; Sahu & Majling, 1993).

The resulting CSA clinker is then interground with up to 35% $CaSO_4$ (including gypsum, hemihydrate or anhydrite or mixtures of them) for optimum setting times, strength development and volume stability (Winnefeld & Lothenbach, 2010; Quillin, 2001a; Glasser & Zhang, 2001; Ioannou, et al. 2014; Ioannou, et al. 2015). By varying the proportions of clinker, calcium sulfate and additions a number of different types of cement can be produced, which are (i) Rapid hardening, (ii) High strength, (iii) Expansive, (iv) Self stressing, and (v) Low alkalinity (which included 15-35% limestone). The role of added calcium sulfate will be highlighted throughout the literature review.

2.4.3 Hydration of CSA cements

When cement is mixed with water, the cement grains begin to dissolve into the water, forming hydrated cement phases around the anhydrous clinker grains. Over time, these clinker grains reduce in size until they are almost gone. The hydrates, consisting of different microstructures (such as needles or platelets), form a solid structure which gain strength with time.

Hydration of ye'elimite ($C_4A_3\hat{S}$)

Immediately after cement wetting, ye'elimite undergoes a rapid hydration reaction which leads to the formation of the main crystalline hydrates within the CSA cement systems, i.e. monosulfate (AFm) and ettringite (AFt) phases, as well as, amorphous aluminium hydroxide (AH_3), as shown in reactions 2-1 and 2-2 (Winnefeld and Barlag, 2010; Glasser and Zhang, 2001). Researchers (Zhang and Glasser, 2002; Glasser and Zhang, 2001) also found that the formation of ettringite begins as early as 15-20 minutes after mixing of cement with water and most of the hydration process occurs within the first 12 hours, suggesting that this is the reason for high early strength gaining of CSA cements.

$$C_4A_3\hat{S} + (16+x)\,H \rightarrow C_4A\hat{S}H_{(10+x)} + 2AH_3 \qquad x = 0, 2, 4 \qquad \text{2-1}$$

$$C_4A_3\hat{S} + 2C\hat{S}H_2 + 34H \rightarrow C_6A\hat{S}_3H_{32} + 2AH_3 \qquad \text{2-2}$$

The extent to which each reaction can occur is largely dependent upon the gypsum content. Initially, ye'elimite reacts with soluble calcium sulfate and water (reaction 2-2) to form ettringite ($C_6A\hat{S}_3H_{32}$), also known as AFt phase. This reaction also takes place with anhydrite or bassanite although at different rates. Formation of ettringite will continue while calcium sulfates are present. In the absence of calcium sulfates, hydration of ye'elimite results to the formation of an AFm type phase with different water contents, depending on the time of hydration, water-to-cement ratio and initial cement phase assemblage (reaction 2-1).

Hydration of belite (C_2S)

In Portland cement systems, hydration of belite leads to the formation of amorphous calcium silicate hydrate (C-S-H) gel of variable composition, as well as, calcium hydroxide (CH), as shown in reaction equation 2-3. However, in CSAB cements, belite coexists with aluminum-rich amorphous hydrates, promoting the formation of strätlingite, as shown in reaction 2-4 (Gartner and Macphee, 2011; Winnefeld and Lothenbach, 2010). Strätlingite is an AFm-type

hydrate containing silica within its anion space. Since belite hydration is very slow the formation of the phase occurs at later ages.

$$C_2S + (2 + x - y)\,H \rightarrow C_ySH_x + (2 - y)\,CH \quad 2\text{-}3$$

$$C_2S + AH_3 + 5H \rightarrow C_2ASH_8 \quad 2\text{-}4$$

The presence of C-S-H phases in hydrated CSA cement is disagreed upon within the literature, which is most likely due to the variations in composition of the particular CSA used. While a number of studies proposed that C-S-H has formed (Zhang and Glasser, 2005; Glasser and Zhang, 2001; Arjunan, et al., 1999), other researchers could not identify its presence within their hydrated CSA systems (Liu, et al., 2015; Winnefeld and Lothenbach, 2010). In any case however, AH_3 gel was found to be consumed during the formation of strätlingite at later ages. CH was not observed as it is incompatible with stratlingite (Andac and Glasser, 1999). Is that absence of CH in hydrated CSA systems that results in lower pH of the pore solution compared to Portland cement systems (Winnefeld and Lothenbach, 2010; Andac and Glasser, 1999).

Hydration of other phases

Since there are several types of CSAB cements, it is not possible to describe every reported hydration reaction. However, the most important reactions of most common minor phases are listed below. Minor phases also react with water or calcium sulfate and somewhat contribute to rapid hardening, according to the following reactions (Meller et al., 2004; Bessy, 1938):

$$C_4AF + 3C\hat{S}H_2 + 30H \rightarrow C_6A\hat{S}_3H_{32} + FH_3 + CH \quad 2\text{-}5$$

$$C_{12}A_7 + 12C\hat{S}H_2 + 113H \rightarrow 4\,C_6A\hat{S}_3H_{32} + 3AH_3 \quad 2\text{-}6$$

$$3CA + 3C\hat{S}H_2 + 32H \rightarrow C_6A\hat{S}_3H_{32} + 2AH_3 \quad 2\text{-}7$$

All these reactions yield AFt and take place during the first hours of hydration. Depending on the availability of calcium sulfates again, there are alternative reactions for the direct hydration of these phases. Most of these reactions also yield aluminum or iron hydroxides (AH_3 or FH_3) which are initially amorphous, but later may crystallize as gibbsite (Cuberos et al., 2010; Morin et al., 2011).

$$C_4AF + 16H \rightarrow 2C_2(A_{0.5}F_{0.5})H_8 \quad 2\text{-}8$$

$$C_{12}A_7 + 51H \rightarrow 6C_2AH_8 + 3AH_3 \quad 2\text{-}9$$

$$2CA + 11H \rightarrow C_2AH_8 + AH_3 \quad 2\text{-}10$$

Setting time

The setting times of CSA cements are shorter than those of PCs and depend mainly on their ye'elimite content and the amount and reactivity of the added calcium sulfate. In the literature values of 30 to 50 minutes haves been reported for initial setting and up to 4 hours for the final one (Zhang et al., 1999; Glasser and Zhang, 2001; G. S. Li et al., 2007; Marchi and Costa, 2011). Furthermore, the setting time can be adapted with the use of retarders (Zhang and Glasser, 2005) and also by the interaction with filler materials (Pelletier-Chaignat et al., 2012).

Role of sulfate content

The hydration process of ye'elimite, as well as, the final performance of CSA pastes, mortars and concretes, mainly depend on the amount and reactivity of the added calcium sulfate (Glasser and Zhang, 2001; Pera and Ambroise, 2004). The solubility of the calcium sulfate added to the clinker influences the ratio of formation of the main hydrate phases, i.e. AFm and AFt (Sahu et al., 1991; Winnefeld and Barlag, 2009; Winnefeld and Barlag, 2010). Increasing amounts of calcium sulfate are associated with formation of higher amounts of ettringite. This can lead to an expansive behaviour of the mixture due to the formation of AFt within the hardened matrix, considering that these cements may contain very large amounts of ye'elimite (Chen et al., 2012). On the other hand, lower amounts of calcium sulfate promote the formation of AFm phases, once sulfate ions have depleted, resulting in reduced fluidity and increased water demand and may also lead to premature setting.

Another important issue is the type of calcium sulfate carrier (i.e. gypsum, bassanite or anhydrite) added to CSA clinker. Setting behaviour can be controlled by the use of a reactive calcium sulfate like gypsum or bassanite which present much higher dissolution rate than anhydrite. Early strength properties are also enhanced by the use of gypsum as sulfate carrier instead of anhydrite (Berger et al., 2011a; Marchi and Costa, 2011; Pelletier-Chaignat et al., 2011). Conversely, low-reactive calcium sulfate mediums, like some anhydrites, cause a lack of calcium and sulfate ions in the pore solution leading to delayed ettringite formation and strength development. The slow dissolution kinetic of anhydrite favours the formation of well-crystallized ettringite particles that leads to rapid loss of plasticity (i.e. lower setting time) and

lower early-age compressive strength (Marchi and Costa, 2011); however, 28 days compressive strength reported to be higher (Winnefeld and Barlag, 2009).

Role of w/c ratio

The w/b ratio required for the full hydration of CSA cements is generally higher than that required for Portland cement (Winnefeld and Lothenbach; 2010) as 32 H_2O molecules are bound according to the chemical formula 2-2. To achieve full hydration of ye'elimite, the stoichiometry determines that a water to binder (w/b) mass ratio of 0.64 is required for reaction 2-2 to proceed, however this w/b mass ratio also depends on the minor phases. Since composition of CSA cements varies considerably, one general theoretical water demand for full hydration cannot be established but should not be far from 0.64.

From an experimental point of view, quite different water-to-binder mass ratios have been employed to study the hydration of CSA cements. These values range from quite low ratios 0.35-0.45 (Chen, et al., 2012; Canonico et al., 2007; Fu et al., 2003; Glasser and Zhang; 2001; Zivica, 2001) to quite high ones 0.70-0.80 (Lura et al., 2010; Winnefeld and Lothenbach, 2010; Winnefeld and Barlag, 2009). The most frequently used ratios range from 0.5 to 0.65, though (Garcia-Mate, et al., 2015; Ioannou, et al., 2014; Buzzi et al., 2011; Valenti et al., 2007; Alesiani et al., 2008; Bernardo et al., 2006).

2.4.4 Mechanical properties of CSA cements.

Strength is a common criterion used to classify cements and also provides a value upon which construction decisions are made. In China, CSA cements are classified by Grades (425, 525, 625, 72.5) in accordance with compressive strength ranging from 42.5 to 72.5 MPa (JC 714, 1996). These cements are traditionally characterised by a rapid setting time and high early strength owed to the rapid reaction of ye'elimite, as mentioned earlier. Although the content and type of calcium sulfate used is vital for controlling the hydration kinetics and dimensional stability (this topic will be further discussed below) of these type of cements, standardised documentation does not correlate strength grades with calcium sulfate additions (JC 714, 1996).

Various research efforts have been made to quantify the influence of calcium sulfate addition on the resulting mechanical properties of both pure ye'elimite and other CSA cement compositions. Many researchers (Hargis, et al., 2014; Trauchessec, et al., 2014; Michel, et al.,

2011) agree that the addition of and increasing gypsum content, at constant w/b ratios, leads to higher strength gains, which is mainly attributed to increasing ettringite formation. Conversely, non-uniform correlations between sulfate and strength at varying w/b ratios have been observed by others. Garcia-Mate et al. (2012) and Ioannou et al. (2014) found that at higher sulfate additions to a CSA cement there was a corresponding decrease in the compressive strengths measured, while Berger et al. (2011) observed that the strength went through a minimum at 10 %wt. addition of gypsum and increased for higher additions. For various compositions of CSA cements with fixed ye'elimite to calcium sulfate ratio, dissimilar effects at low and high w/b ratios have been observed (Beretka, et al., 1996) suggesting that an optimisation is required around a given ye'elimite content.

CSAB cements have also the advantage of developing higher long term strength due to the formation of C-S-H gel or stratlingite at later ages (Quillin, 2001). As stratlingite forms as hexagonal platy crystals and due to its low density (1.9 g/cm^3) it can have good space filling properties (Okoronkwo and Glasser, 2016).

2.4.5 Dimensional stability of CSAB cements and theories of expansion

CSA cements have a reputation for being expansive in nature. This is mainly attributed to the formation of ettringite, as it has a lower density and greater space filling capability than many other cement phases; however, the lack of understanding of the exact link between these two processes have generated considerable debate and controversy in the research community.

Dimensional stability in CSA systems has been shown to be dependent on the amount of calcium sulfate (Bizzozero, et al., 2014; Chen, et al., 2012; Glasser and Zhang, 2001, Yanmou, et al., 1992) available in the system, with higher additions linked to elevated ettringite formation and greater potential of expansion to occur. For instance, Yanmou et al. (1992) using a CSAB clinker that contained 58.4% $C_4A_3\hat{S}$ and 11.6% C_2S observed approximately 0.15% and 0.70% expansion with 15% and 35% $C\hat{S}H_2$, respectively. In another study, Glasser and Zhang (2001) reported that a different CSAB cement (unreported phase assemblage) shrunk slightly at a $C\hat{S}H_2$ addition below 22%, showed virtually zero dimensional change at 22-24% $C\hat{S}H_2$ addition, and became expansive at additions of 24% and higher. They concluded that in order to achieve a dimensionally stable CSA cement, the added calcium sulfate should be chosen to be slightly below the theoretical amount required to maximise the ettringite content.

Regardless the molar ratio of calcium sulfate to ye'elimite, the ye'elimite content of the cement itself can increase the potential formation of ettringite (Beretka, et al., 1996).

Others researchers (Telesca, t al., 2014; Mehta, 1973) agree that, the ettringite formed during CSAB hydration can be either expansive or non-expansive depending on whether the reaction takes place in the presence or in the absence of calcium hydroxide, respectively, as shown in reaction 2-11, where three moles of ettringite can form from one mole of ye'elimite.

$$C_4A_3\hat{S} + 6CH + 8C\hat{S} + 90H \rightarrow 3C_6A\hat{S}_3H_{32} \qquad 2\text{-}11$$

Except the extent of ettringite presence in the hydrated system, the time of its formation has also been found to effect expansion in CSAB. There is a general agreement that ettringite formation prior to final setting of the cement is non-expansive, while that formed later within the hardened cement matrix (secondary or delayed ettringite) leads to expansion and possible cracking. A situation like that can be observed at low w/c ratios, where high ye'elimite-bearing CSAB cement can undergo self-desiccation because the formation of ettringite requires large amounts of water. As a result, large amounts of cement particles may remain unhydrated after setting, even at later ages (Glasser and Zhang, 2001). This effect can lead to an expansion behaviour if the cement is later exposed to water from the external environment, as unhydrated phases can react to form secondary ettringite (Beretka et al., 1996; Older and Colan-Subauste, 1999). Similarly, low calcium sulfate to ye'elimite ratios, below the stoichiometric ratio required for the full hydration of ye'elimite, promote, in the presence of an external source of sulfates ions diffusing inside the hydrated cement, the transformation of monosulfate, previously formed, into ettringite (Whittaker and Black, 2015).

While the theory of an increase in solids volume due to ettringite formation appears to be an obvious explanation for expansion to occur, and is in fact a pre-requisite; it is not alone sufficient to account for expansion and there is a lack of evidence to link expansion directly to ettringite formation (Kunther, et al., 2013; Taylor, et al., 2001; Older and Colan-Subauste, 1999). For instance, the formation of other phases such as CAH_{10} also results in an increase of the overall solid volume, but its formation is not generally considered to be expansive (Bizzozero, et al., 2014).

Mehta (1976) proposed that expansion occurs due to the swelling pressure, which he attributed to the formation of a colloidal form of ettringite, which presents in a gel-like form. This form

of ettringite would then be able to absorb water molecules within its structure, causing expansion. However, he gave little theoretical support for this hypothesis. Besides that, it is unlikely that the ettringite in hydrated cement pastes would form with gel-like properties as they tend to exhibit a very crystalline form; a crystalline structure cannot absorb water molecules within its crystal lattice and therefore cannot swell (Bizzozero, et al., 2014; Kunther, et al., 2013; Yu, et al., 2013).

The most plausible mechanism of expansion is based on the idea of crystallisation pressure. As discussed by Scherer (2004a,b; 1999), there are two necessary conditions for expansion to occur by such a mechanism. First the crystal must be growing from a supersaturated solution, which provides the energy for expansion, according to reaction 2-12. Second, the crystal must grow in a confined space and exert the expansive force on the walls of the pore of the material in which it is forming. The physics of this process have been presented in detail by researchers (Scherer, 2004a,b, 1999; Flatt and Scherer; 2008) who showed that there is an inverse relationship between the pressure exerted and the size of the pore. Therefore typically crystals growing in small pores (of the order of tens of nanometres) are likely to be most significant in terms of generating expansive pressures. The solution concentration plays a critical role as it determines the minimum pore size in which crystals can grow. Therefore with higher supersaturation a larger pore volume is accessible to growing ettringite crystals exerting pressure in the porous skeleton.

$$\Delta p = \frac{RT}{V_m} ln \frac{Q}{K} \qquad 2\text{-}12$$

Where p is the pressure exerted by the crystal growth, R is the gas constant, T is the absolute temperature, V_m is the molar volume of the crystal, Q is the ion activity product and K is the theoretical equilibrium solubility product of the crystal.

Although experimental measurements of the pore solution concentrations at various points within the cement matrix are difficult to obtain, this theory is the most agreed upon within the literature (Whittaker and Black, 2015; Bizzozero, et al., 2014; Yu, et al., 2013; Ping and Beaudoin, 1992). Bizzozero et al. (2014) suggested that increasing the calcium sulfate content leads to a higher saturation of the pore solution with respect to ettringite, thereby the conditions required to cause crystallisation pressure can be met. This could explain the critical amount of gypsum leading to high unstable expansion.

2.4.6 Microstructure and Porosity

Permeability has a key role in concrete durability, as it controls the rate of flow of deteriorating agents into the concrete matrix. The permeability is highly influenced by the pore structure of the concrete. Although the total porosity of a concrete will influence its permeability, the pore size distribution and the interconnectivity of pores is of greater importance. Since most of the aggregates used in concrete production tend to have few interconnected pores and, it is mostly the pore structure of the cement paste which governs the permeability of concretes to water or ions (Page and Page, 2007).

The hydrated phases formed within a cement paste play an important part in determining the porosity and pore structure. In CSA cements, the two main hydrates are ettringite and monosulfate, which have dissimilar morphological characteristics. Ettringite is characterised by elongated needle like shape crystals, while monosulfate shows hexagonal platelet features (Figure **2.10**). In addition to having a lower density, 1.78 gcm^{-3} (Balonis and Glasser, 2009), than many other hydrates in cements, ettringite has been suggested to have good space filling properties in CSA systems due to the dense packing of the individual crystals (Glasser and Zhang, 2001).

In CSA systems, the rapid formation of ettringite in large amounts is capable of quickly reduce the internal pore space; thus a bimodal pore structure is developed earlier than in Portland cement systems (Telesca, et al., 2014; Garcia-Mate, et al., 2012; Bernardo, et al., 2006; Glasser and Zhang, 2001). A bimodal pore structure generally correlates to a disconnected pore structure and a denser microstructure, which both could be potentially beneficial in marine environments as a bimodal pore structure suggests disconnected and isolated pores (Telesca, et al., 2014). However, it has been suggested that this could also contribute to expansion due to the limitation of pore space in the case of delayed ettringite formation occurring (Chen, et al., 2012). Similar to the strength, both the calcium sulfate and water content play a role in the pore structure development of CSA cements, higher w/b ratio leads to higher pore space, as with Portland cements, while the influence of gypsum content is variable (Telesca, et al., 2014; Garcia-Mate, et al., 2012).

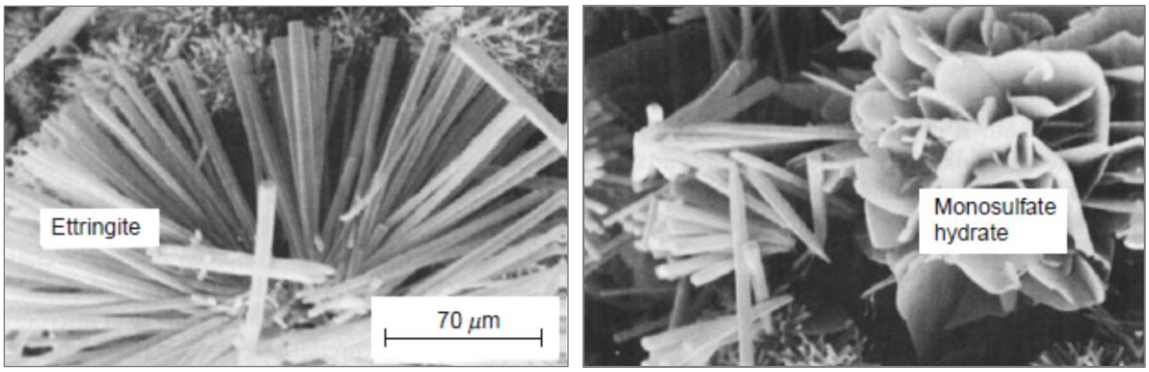


Figure 2.10: SEM images showing typical ettringite needles and hexagonal monosulfate platelets, reprinted from (Quillin, 2001).

2.4.7 Alkalinity of CSA pastes, mortars and concretes

The alkaline environment of PC (pH of 12 to 13) provides embedded steel with corrosion protection by means of a thin oxide layer. This passive film does not actually stop corrosion but reduces the corrosion rate to an insignificant level. The critical pH value for steel reinforcement passivation layer is slightly above 11.5. Below this pH level, the protective oxide film on the steel surface starts to decompose. Therefore, the alkaline nature of ye'elimite-containing cement pastes is a key issue for its durability.

Data on the pore solution composition and alkalinity of CSA binders is very scarce in literature. Moreover, the large compositional variability in CSA cements, may results to large differences in pore solution properties. The ability of CSA cements to generate internal pH values upon hydration in excess of the steel passivation threshold has been established by several authors in the literature.

Andac and Glasser (1999) studied the pore solution chemistry of a CSA clinker at w/c ratios of 0.5 to 0.8 and at different hydration ages (from 1 to 60 days) and found a rapid release of soluble alkalis from the clinker accompanied by a high pH, ~13. In a more recent study, Winnefeld and Lothenbach (2010) presented a very comprehensive pore solution chemistry and alkalinity study of the hydration of two CSA cements, at similar conditions i.e. w/c ratios of 0.72 and 0.80 and hydration periods between 1 and 28 days, respectively. They found that during the first hours of hydration the pore solution of both cements is dominated by potassium, sodium, calcium, aluminum and sulfate, which result to relatively low pH values, between 10 and 11; however, after approximately one day and when the calcium sulfate is depleted, the sulfate concentration drops by a factor of 10 and the pH is increased to

approximately 12. At 28 days of hydration, both CSA cements gave much higher pH values, between 12.5 and 13. In contrast, Janotka et al. (2003) reported pH values of a CSAB cement of 10.3 to 10.5, which were insufficient for full steel passivation.

As shown above, alkalinity of CSA cements is generally about 1 pH unit lower than that of Portland cements, particularly at early ages. The experimental results showed above, as well as, performance data obtained from CSA concretes exposed for 14 years in the intertidal zone (Glasser and Zhang, 2001) suggest that CSA cements can provide enough protection to the embedded steel; however, regimes of corrosion and of protection still need to be quantified and related to engineering practice.

2.4.8 CSA blends with supplementary cementitious materials

Although the eCO_2 associated with the CSA cements may be lower than that of PC, potentially greater savings may be achieved based on the development of CSA blends with supplementary cementitious materials (SCMs) with maintained performance properties. Blending of Portland cement with low eCO_2 pozzolanic by-products is a well-established practise to produce sustainable concrete with improved performance.

In Portland cement systems, the calcium hydroxide needed for pozzolanic reaction to take place is produced during the hydration of belite and mainly alite (Hewlett, 2012). The extent of pozzolanic reaction and its effects on the properties of the blend is affected by the content of both phases in Portland cement. In CSAB systems, belite is the only source of calcium hydroxide production. Owing to the composition of belite, having lower calcium oxide content comparing to alite, the production of calcium hydroxide during hydration of CSAB cements is significantly decreased. Therefore, the optimal portions of pozzolans added to CSAB cement blends should be expected to be lower from those in Portland cement systems, which usually accounts for 20% to 40% by mass of cement.

To the best of my knowledge, there is little work on the role of mineral additions to partly replace CSA or CSAB cements. Zivica (2000, 2001) who studied the pozzolanic reactions of CSAB blends with fly ash, silica fume and slag at replacement levels up to 3O% by mass of cement concluded that an equal or improved performance can be achieved by pozzolan additions in the range of 5 % to 15% by mass. More recently, Guo and Shi (2016) and Ioannou et al. (2014) in similar studies found improved mechanical and durability performance of

CSAB blended systems containing small portions (up to 15% by mass) of SCMs. Finally, they all conclude that the optimum replacement portion should be decided in a case-by case basis.

Still there is limited understanding and lack of experimental data on the hydration mechanisms of such systems. Given this, and by considering the advantageous effect of particle packing that pozzolanic materials may potentially provide when acting as a low eCO_2 filler, then it is possible that a more sustainable system may be developed whilst maintaining its mechanical and microstructural properties.

2.4.9 Properties of pastes/mortars/concretes made with CSA

Fresh properties

Rheology of fresh paste/mortar/concrete is often characterized by its workability. In general terms, CSA mixes exhibit reduced fluidity and shorter setting times compared to those of PC mixes for equal w/c ratios due to rapid hydration of ye'elimite. As highlighted previously, this sulfoaluminate phase is characterized by a high water demand, as 34 H_2O molecules are bound according to the chemical formula (reaction 2-2), and its hydration results almost instantly to the formation of a skeleton network of ettringite crystals. Similarly to PC mixes, special admixtures (retarders and/or superplastisizers) are usually needed to ensure that the CSA concrete will remain workable and compactable over a sufficient period of time (Ioannou, et al., 2015; García-Maté, et al., 2012; Chang, et al., 2009; Quillin 2001).

Other researchers considered the use of fly ash addition as a mean to enhance workability of CSA cements. García-Maté et al. (2013) suggest that CSA pastes blended with small additions of FA, up to 15%, showed lower viscosity values from only CSA pastes for the same w/b ratio. However, in terms of setting time, Ioannou et al. (2014) by replacing CSAB by fly ash (15% by mass) noted slightly reduced final setting and no effect on initial setting time of paste. Likewise, Zivica (2001; 2000) observed negligible effect of small portions (5-15%) of fly ash or ggbs on initial and final setting of a CSAB.

Quillin (2007; 2001) has tested a wide range of CSA formulations and CSA blends and observed that CSA concrete mixes tend to show a premature stiffening within 5 to 10 minutes after mixing. In most of the cases, remixing of concrete for a short period was sufficient to return it to a workable state, but the loss of slump with time was greater compared to the PC mixes, particularly at low w/c ratios. Ioannou et al. (2015) also measured the loss of slump of

a CSAB/FA combination over a PC reference concrete and the behaviour observed was in line with the early results given by Quillin (2007; 2001).

Engineering properties

Compared to PC, CSA cements exhibit faster early strength development. This is well demonstrated in a number of studies (Quillin, 2001; Ioannou, et al., 2015). For instance, in the study of Ioannou et al. (2015) the CSAB/FA mix at all w/c ratios reached almost 85-95% of their 28 days' strength after just 7 days. The CSAB/FA concrete continued to gain strength beyond 28days, with the strength at 180days be approximately 6-8% higher than the 28-days strength, possibly due to the fly ash contribution and the slow hydration of the belite phase (C_2S content was around 15%). As mentioned earlier, the existence of belite in the cement matrix is expected to provide the long-term strength development. This was also confirmed from Quillin (2001; 2007) who observed a continued increase (~25%) in the strength of concrete made with a commercial CSAB cement for up to 2 years. However, the strength development and volume stability depend on the type and amount of the calcium sulfate added (Alaoui, et al, 2007; Péra, et al, 2003).

One of the main advantages of CSA compared to PC is its unique shrinkage compensation property, which was the initial reason to develop this type of cement (Klein, et al, 1961; Telesca, et al., 2014). This is due to the formation of an expanding component i.e. ettringite crystals, and the high water demand required for hydration of ye'elimite – hence most of the mix water is consumed for hydration and less excess water is available to evaporate and cause plastic or drying shrinkage cracking. These issues has been discussed extensively in the 'Hydration process' and 'Dimensional stability' sections above.

Durability performance

Durability properties are the most important characteristics of concrete, as they are closely associated with the service life of concrete structures. As mentioned previously, the permeability of the concrete has a key role on durability, since in most cases a deteriorating agent diffuses into the cement matrix and slowly cause disintegration, either by alter its chemical composition or by forming expansive products. However, the importance of "time scale" has been highlighted with respect to this (Feng, et al., 2015; Neville, 2012). Indeed, permeability is the major factor that control durability performance of concrete structures with

an intended service life up to 15 years. For concrete structures with an intended service life of 25 to 100 years, low permeability may not be sufficient, and in this case, the cement composition and chemical modifications to it become of greater importance, as these will govern the extent of attack.

The sorptivity properties of concretes made with CSA/FA, PC and PC/GGBS cements were determined by Ioannou et al. (2015) according to ASTM C1585 (ASTM, 2013) method. The results indicated that the CSA blend resulted in significantly lower values than that of PC and similar to that of PC/GGBS.

In general, good sulfate resistance of CSA cements has been noted in the literature, but with very little provided in the way of understanding of the mechanisms which may occur. Existing theories on the reactions occurring in these environments have yet to be applied to CSA cement matrices and their performance requires further research. Monteiro et al. (2000) tested the sulfate resistance of a range of cements, including two CSA compositions, by means of change in compressive strength of cement paste cubes fully immerged in 4% Na_2SO_4 solution for a range of exposure periods; while one of the CSA cement performed admirably, the other was found to be the poorest performing of all the cements tested. Chen et al. (2012) in a study investigating the expansive nature of CSAB they observed almost zero expansion of CSAB cement pastes submerged to concentrated Na_2SO_4 solutions. The damage of CSA pastes partially immersed in highly concentrated magnesium sulfate solutions was also studied by Liu et al. (2015). The authors expected that in the absence of calcium hydroxide in CSA cement pastes no reactions would take place between the cement and externally introduced sulfates; however, they noted that in the submerged section extensive spalling, decomposition of ettringite and precipitation of gypsum and calcium carbonate had taken place.

Experimental data on sulfate performance of CSA concretes are very rare in the literature. Quillin (2001) suggest that concretes made CSAB cements exhibit excellent sulfate resistance, however he based that conclusion only to visual and qualitative XRD observations i.e. the ettringite levels were increased without effect on the compressive strength of concrete. In a more recent study, Ioannou et al. (2015) monitored the length change of concrete samples made with a CSA/FA cement combination. The prisms were fully submerged in a 5% Na_2SO_4 for a period of 40 weeks, and the measurements showed that the CSA/FA combination featured the lowest expansion at all w/c ratios throughout the test period compared to the PC

based reference mixes. This behaviour was a result mainly of the "aluminate-bearing phases being bound in the ettringite phase and unavailable for reacting with external sulfates to form expansive sulfoaluminates in the hardened paste". However, this explanation can only stand in situations where the added calcium sulphate content was high enough to convert all the AFm phase to AFt at very early ages (Zhang and Glasser, 1999). It needs to be kept in mind that the optimum gypsum content for potentially sulfate resisting CSA cement should not compromise at the same time any of the other properties of the cement. The addition of fly ash in the mix also resulted to a denser microstructure (reflected through lower absorption rates) but its contribution with respect to sulfate resistance requires further investigation.

Ettringite was reported to be susceptible to carbonation (Bertos, et al., 2004) and therefore it has been suggested that, due to the large amount of ettringite produced on hydration, concretes made with CSA cement may be particularly prone to carbonation. This process could lead either to deterioration in the concrete itself (through decomposition of ettringite and changes in the solid volume of the cement paste) or to reinforcement corrosion (through the loss in pH buffering capacity).

Glasser and Zhang (2001) and Zhang and Glasser (2005) by carrying out both laboratory experiments and field measurements in CSA concretes removed from service, respectively, concluded that carbonation rates in CSA cements are comparable to those in PC based materials of comparable quality. However, limited explanation of this was given and the initial conditions and mix design of the concretes analysed being unknown. In contrast, Quillin (2001) found that CSAB concretes carbonated more readily than a comparable PC concrete, which had a similar cement content and had been similarly treated and even faster from a 50:50 PC/FA concrete with similar strength. In line with these are also the results obtained from accelerated carbonation tests carried out by Ioannou et al. (2015), where CSA/FA concrete not only showed higher carbonation depth than both PC and PC/GGBS concretes, but also increased faster with an increase in w/c ratio. The authors found that the dense microstructure developed by ettringite and firmly accommodated fly ash particles is not a parameter that significantly influences the resistance to carbonation as this particular behaviour suggest that the carbonation-impeding alkalis in the pore solution may underpin an equally significant role in carbonation resistance.

In a previous study concerning the hydration products of CSA/FA paste, Ioannou et al. (2014) suggested that little or no $Ca(OH)_2$ is available in the combination to impede carbonation and thus the predisposition of ettringite to carbonate is high. Therefore, the carbonation resistance of the concretes would not only be strongly defined by the porosity and diffusivity aspects of concretes but also by the presence of alkali hydroxides. This conclusion is of high importance, as the hitherto practice of using the compressive strength in Portland cement concrete as an indirect means of assessing its carbonation resistance would be misleading in the case of a CSA/FA concrete because of such predisposition of the combination.

The ability of CSA cements to generate internal pH values upon hydration in excess of the steel passivation threshold has been presented earlier. Less reported is the important aspect of the maintenance of said passivation to ensure attainment of designed service lifespans. In PC systems, the clinker hydration is accepted to result in an AFm phase (monosulfate), which is known to bind chlorides via conversion to Friedel's salt (Chudek, et al., 2000; Suyavanshi, et al., 1996). This binding capacity can be further enhanced by the use of both fly ash and slag (Luo, et al., 2003; Dhir, et al., 1996).

In CSA systems, the capacity of hydration products to bind chlorides has been well demonstrated by Paul et al. (2015) with respect to a CSA/PC blend while in contrast a pure CSA having 16 weight percent calcium sulfate (equivalent to a molar ratio of anhydrite to ye'elimite of 1.8) had virtually no ability to bind diffusing chloride ions. As the production of solely AFt (ettringite) from ye'elimite hydration requires 2 moles of calcium sulfate for each mole of ye'elimite, it was suggested that the correlation of chloride binding in blended cements is owed to a deficiency of sulfate content in the blend which alternatively promotes AFm formation. The role of hydration speciation influence might also explain the poor laboratory performance, relative to a reference PC, experienced by a cement mixture over-dosed with calcium sulfate investigated by Kalogridis et al. (2000) in which steel mass losses were accelerated in a chloride solution exposed sulfoaluminate matrix. Similarly, the high ettringite content matrix investigated by Quillin (2001) may be explained to have a high chloride diffusion coefficient in 90 day marine exposure testing due in part to reduced binding potential. Dachtar (2004) has also studied the chloride permeability through electrical conductivity of CSA concretes and found that they exhibited less electrical resistivity due to their low alkali content and lower pH values compared to PC concretes.

On the contrary, Ioannou et al. (2015) for a given w/c ratio observed lower chloride diffusion coefficient for the investigated CSA/FA combination over the reference PC concrete, but slightly higher for the PC/GGBS concrete. According to the authors, this effect can be attributed to the beneficial effect or fly ash incorporation with respect to resulting in a denser microstructure which moderates the ingress rate of chlorides and the likely contribution of ettringite phase to bind chlorides. About the latter, there are different findings in the literature that assign the phase either with a low or with a high binding ability (Ekolu, et al., 2006; Hirao, et al., 2004). Nevertheless, the authors concluded that the risk of chloride-induced corrosion for the CSA/FA cement combination was comparable to PC. Likewise, CSA concrete pipes after 14 years' service in the intertidal zone were found by Glasser and Zhang (2001) to exhibit no reinforcement corrosion even in areas where the concrete cover was only 7-8 mm.

2.4.10 Conclusions

Developed in China in the 1970's, calcium sulfoaluminate cements are a class of specialty cements that are included in the family of rapid-setting cements. Stand alone as a cementitious binder or blended with PC, CSA have been used on a large scale in China for special applications where rapid hardening and high early strengths were need, to achieve shrinkage compensating concrete or to encapsulate radioactive element / hazardous waste in high-density cement matrixes. Outside China the use of CSA cements are limited due to lack of a robust internationally recognized regulatory standard.

In comparison to Portland cement, CSA cement can achieve energy savings as high as 25% and provide environmental benefits by reducing CO_2 emissions by around 20%. If however an alternative cementing system is ultimately to have a real impact on global CO_2 emissions related to the construction industry, it has to perform at least as good as the current generation of Portland-based cements, and probably even better, because it is likely to be, at least initially, more expensive to the consumer.

The mechanical performance and durability of building materials made from CSA cements, as derived from laboratory tests and field studies so far, seems to be in general at least comparable to conventional Portland cement-based materials. However more data concerning long-term behaviour of this type of cement and particularly in aggressive environments are needed before the introduction of appropriate new standards and construction codes.

CHAPTER 3: PROGRAMME OF WORK, MATERIALS AND EXPERIMENTAL DETAILS

This Chapter provides details of the experimental programme and the test procedures followed during the study. These include general descriptions of the test methods used to characterise the properties of recycled aggregates, assess the mechanical and durability performance of a calcium sulfoaluminate cement, as well as procedures for mixing and testing fresh and hardened concrete.

3.1 Programme of Work

The programme of work comprised a literature review and four main stages, as shown in Figure 3.1 and described in the following sections.

3.1.1 Phase 0 – Literature Review

Initially, a comprehensive literature review was carried out to examine several important issues in relation to the properties of recycled aggregates and calcium sulfoaluminate belite cements and their effect on fresh, engineering and durability properties of concrete. Their environmental impact was also considered. In this phase, a wide range of documents from various sources (e.g. standards, journal articles, books, etc) were collected and examined. Potential techniques were also identified for inclusion in the experimental programme.

3.1.2 Phase 1 – Material Characterisation

Coarse and fine recycled aggregates typically produced in Qatar were obtained throughout a year period to provide comprehensive coverage of aggregate properties. These included recycled aggregates from processed construction and demolition wastes (CDWA) and recycled aggregates from processed excavated wastes (EWA). Alongside with recycled aggregates, natural coarse and fine aggregates that are representative of those used by the construction industry in Qatar were also provided as benchmark components.

In this stage, the obtained recycled aggregates were characterized physically and chemically, following the recommendations of BS EN 12620 (BSI, 2002). This included composition, grading, particle density, water absorption, particle shape, resistance to fragmentation and drying shrinkage tests to assess the physical properties of the materials, while LOI, alkali-silica reaction, water and acid soluble sulfates, x-ray florescence (XRF) and x-ray diffraction (XRD)

tests were used to determine the chemical properties of the materials. Properties such as resistance to freezing and thawing or resistance to surface abrasion where not tested either because were not applicable to the environment of Qatar or were not agreed with the client during the project planning stage. The results of these tests were then used to make comparisons with reference aggregates, to establish conformity with the limits set both in British and Qatar standards and to determine their suitability for use in concrete.

A commercial strength grade 72.5 MPa rapid-setting CSA-belite (CSAB) cement containing a high ye'elimite content and no calcium sulfate was also obtained from China to replace CEM I in selective concrete mixes. The physical, mineral and chemical characteristics of the CSAB were provided by the supplier and verified by our collaborative lab in University of Aberdeen. Prior its use in concrete work, the mechanical and durability characteristics of CSAB cement were assessed in relation to elevated amount of added gypsum. This led to the identification of an optimum gypsum content that provides prolonged setting time, maximises strength, controls expansion and enhances chloride and sulphate resistance.

3.1.3 Phase 2 – CEM I-based recycled aggregate concrete

Having established the characteristics of CDWA and EWA, the study progressed to determine the effect of these aggregate types on fresh and hardened properties of concrete. In the preliminary stage, trial mixes were carried out to determine admixture doses to control concrete consistence (target slump: S3). Four w/c ratios (0.45, 0.55, 0.65 & 0.75) were considered to provide a range of practical mixes and strengths. Recycled aggregates levels were fixed at 30%, 50% and 100% (by volume of reference aggregates) for concrete mixes incorporating CDWA and at 15% and 30% for those containing EWA. The majority of the concrete mixes were produced with a CEM I 52.5N cement. Selective mixes were also produced with binary cement combinations comprising a blend of CEM I with type-S fly ash (40% w/w) or GGBS (50% w/w). These addition levels were relatively high and selected to provide a substantial reduction in eCO_2 of concretes. The framework of mixes used, enabled interpolation and comparisons to be made with respect to specific parameters of the test concretes, e.g. equivalent strength.

Measurements were made of concrete consistence, plastic density, strength development, drying shrinkage, permeation properties (initial surface absorption) and aspects of concrete durability (chloride ingress, carbonation and sulphate resistance).

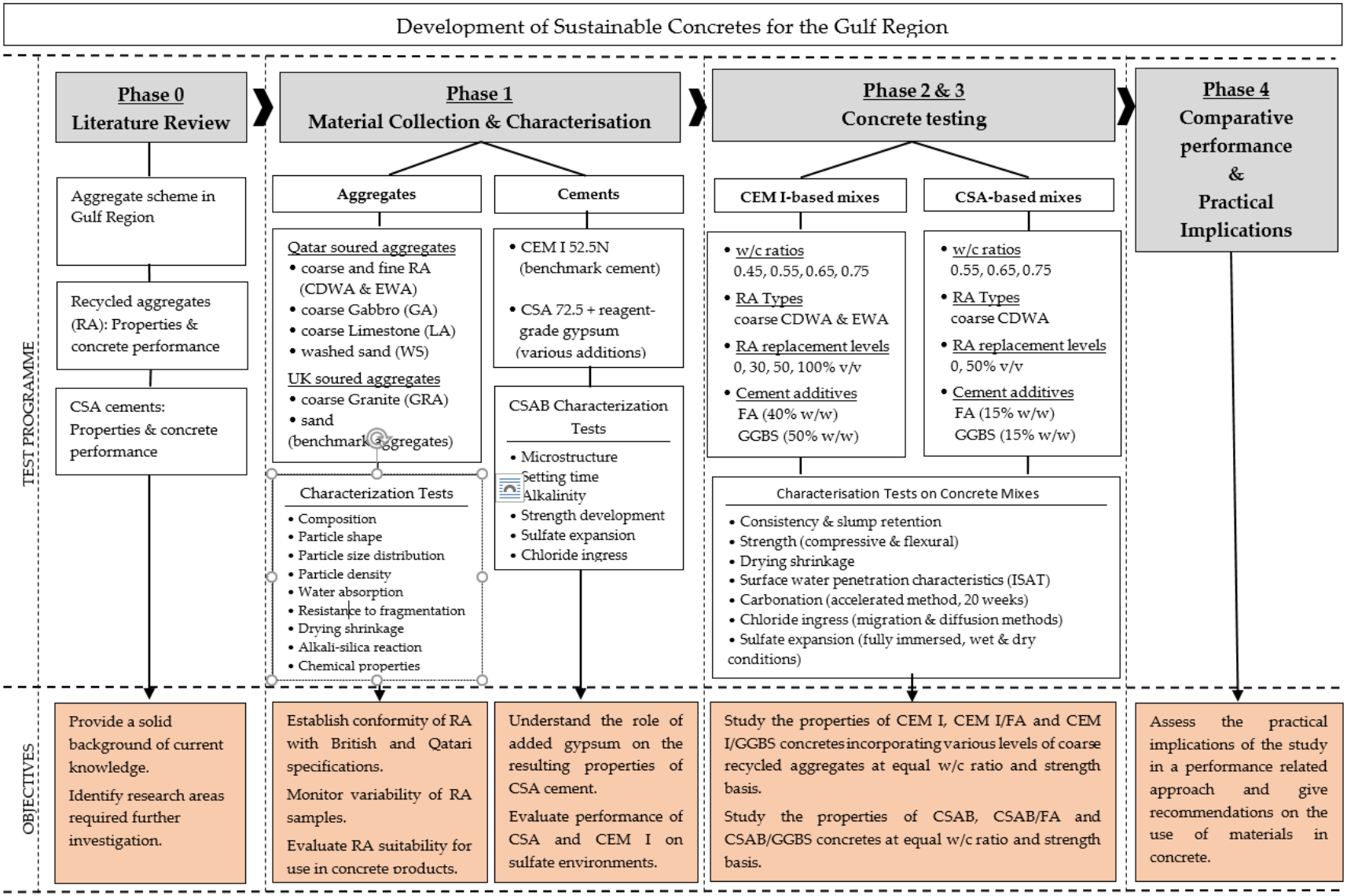


Figure 3.1: Overview of the experimental programme

3.1.4 Phase 3 – CSAB-based recycled aggregate concrete

Selective concrete mixes from phase 2 were tested in phase 3 using CSAB cement. The w/c ratios considered here were 0.55, 0.65 and 0.75 enabling the properties of the CSAB-based concretes to be compared with those of CEM I-based concretes at an equal strength basis. Initially, trial mixes were carried out to determine superplasticizer doses and achieve an S3 slump. Only CDWA were used at a replacement level (by volume of reference aggregates) of 50%. A few mixes were also produced with CSAB blends with 15% w/w FA or GGBS. These levels were substantially lower form those used in CEM I mixes following literature findings.

Similarly to phase 2, the concrete consistence, slump retention, plastic density, strength development, drying shrinkage, permeability (initial surface absorption), chloride ingress, carbonation and sulphate resistance were measured.

3.1.5 Phase 4 – Assess the practical implications of the study provide guidance on the use of recycled aggregates and CSA cement in construction

The outcomes of the work were used to establish the influence and impact of the materials considered in this study in properties of equal strength concrete. These are discussed in the final part of the study, with the aim of providing guidance with regard to the use of these materials in construction and potential reduction of the embodied CO_2 of concrete.

3.2 **Materials**

3.2.1 Aggregates

A total of 26 No. samples of recycled aggregates were sourced from the largest construction and demolition waste treatment area in central Qatar and shipped to UK for inclusion in the project. The samples comprised (i) aggregates from processed construction and demolition wastes (CDWA) and (ii) aggregates from processed excavated wastes (EWA). These are the two recycled aggregate types specified in QCS 5-2 (QS, 2014) to be used by the construction industry in Qatar. The recycled aggregates were graded into three size fractions i.e. 0/4, 4/10 and 10/20. Sampling was carried out in accordance with BS EN 932-1 (BSI, 1997) throughout a year period so that representative quantities to be obtained from the aggregate stockpiles. This also allowed to monitor the variability of the materials with time. Information relevant to the

origin of parent material, the volume of pre-existing stockpiled aggregates and the processing systems implemented by the recycling plant was not provided.

In addition to recycled aggregates, samples of construction aggregates typically used in Gulf Region i.e. coarse gabbro and limestone aggregates imported from UAE and a local washed dune sand were also supplied from a major local readymix company to be used as benchmark materials. These quantities, however, were not sufficient to cover the project requirements in reference aggregates and therefore crushed coarse granite aggregates and a construction sand, both conforming to BS EN 12620 (BSI, 2002), were sourced from a UK quarry.

The details of all aggregates, as well as, its physical, mechanical and chemical properties are given in Chapter 4.

3.2.2 Cement and cement constituents

A PC (CEM I 52.5N class) and a commercial Grade 72.5 Rapid-setting CSA-belite (CSAB) cement containing a high ye'elimite content and no calcium sulfate were used in this study. A type-S Fly Ash (FA) and a Ground Granulated Blastfurnace Slag (GGBS) were also considered in binary cement combinations to reduce embodied CO_2 and enhance durability of selective concrete mixes. The CEM I was conforming to BS EN 197-1:2000, CSAB to GB20472 (SAC, 2006), FA to BS EN 450-1 (2005) and GGBS to BS 15167-1 & 2 (2006).

All materials were obtained in dry powder form from UK sources with the exception of CSAB which was sourced from China and stored in plastic airtight containers until use to prevent deterioration. The mineral phase and oxide compositions, as well as the physical properties of the materials as provided by the suppliers are given in Table 3.1 and Table 3.2.

Table 3.1: Mineral composition of cements and cement constituent materials (% by mass)

CEM I	CSAB	Fly Ash
Alite, C_3S (59.5)	Ye'elimite, $C_4A_3\$$ (53.6)	Quartz (6.5)
Belite, C_2S (15.5)	Belite, C_2S (20.6)	Hermatite (2.0)
Tricalcium Aluminate, C_3A (8.5)	Ghelenite, C_2AS (11.7)	Magnetite (0.1)
Tetracalcium aluminoferrite, C_4AF (8.8)	Perovskite, CT (4.8)	Mullite (9.3)
Gypsum (4.9)	Merwinite, C_3MS_2 (4.2)	Glassy/Amorphous (77.7*)
	Mayenite, $C_{12}A_7$ (3.3)	

* Calculated value (LOI has been substracted from total)

Table 3.2: Physical properties and bulk oxide composition of cements and cement constituent materials

	CEM I	CSAB	FA	GGBS
Physical properties				
Specific surface area (BET method), m^2/g	1.45	1.43	2.12	-
Blaine, cm^2/g	-	≈ 6000	-	-
Particle density, kg/m^3	3.13	2.91	2.34	2.95
d10, μm	3.3	2.2	2.1	1.8
d50, μm	17.5	15.5	14.5	11.7
d90, μm	54.3	64.2	69.9	35.2
LOI, %	-	-	4.4	-
Fineness (45 μm sieve), %	-	-	11.5	-
Oxide composition				
Calcium oxide (CaO), %	63.8	44.7	4.1	39.6
Silicon dioxide (SiO_2), %	20.7	11.0	48.2	34.4
Aluminium oxide (Al_2O_3), %	4.7	32.1	23.1	13.3
Iron oxide (Fe_2O_3), %	2.7	2.3	8.5	0.36
Magnesium oxide (MgO), %	1.1	1.3	1.8	7.9
Titanium dioxide (TiO_2), %	0.5	1.5	1.1	0.6
Potassium oxide (K_2O), %	0.6	0.3	2.8	0.7
Sodium oxide (Na_2O), %	0.3	0.1	1.3	0.4
Phosphorus pentoxide (P_2O_5), %	0.2	-	0.5	0.0
Sulfur trioxide (SO_3), %	3.1	7.1	0.8	1.1

To control the properties of the CSAB cement, a commercial gypsum, having an SO_3 content of 51.2% w/w and a Blaine fineness of ~5800 cm^2/g was used. It is important the particle size of the gypsum to closely match that of the cement as this can have an impact on the hydration kinetics (Berger, et al, 2011). A type RFC-1 retarder compatible with CSA cements was sourced from CSAB supplier to be used in the study. The retarder had a pH value of about 10 and a fineness of 10% maximum residue on 0.08mm sieve. The recommended quantity was in the rang of 0.5 to 2.0 % by weight of cement.

3.2.3 Superplasticizing Admixtures

A high-range water-redusing/superplasticising admixture based on polycarboxylic ether polymers and conforming to BS EN 934-2 (BSI, 2009) was used throughout this study to

achieve the required consistence of concrete mixes. This was a brown liquid with a specific gravity of 1.09 g/cm^3, pH value of 6 ± 1, chloride content of less than 0.10% by mass and alkali content of less than 1.0% by mass (information provided by the manufacturer).

3.2.4 Chemical Reagents

An analytical reagent grade sodium chloride (99.99% NaCl) and calcium hydroxide ($Ca(OH)_2$) standard laboratory grade were used as required with chloride testing procedures. The sulfate exposure solution was prepared using a standard laboratory grade sodium sulfate (99%, extra pure, anhydrous, Na_2SO_4). Other chemical reagents were used as specified in the test procedures.

3.2.5 Water

Potable tap water, in accordance with BS EN 1008 (BSI, 2002) was used to cast all paste, mortar and concrete mixes. This was also used for curing and testing of mortar and concrete specimens. Double distilled and de-ionised water was used for all the analytical laboratory work.

3.3 **Characterisation tests on aggregates**

For the characterisation of physical, mechanical and chemical properties of aggregates, representative test portions were obtained from each aggregate sample in accordance with BS EN 932-1 (BSI, 1997) and BS EN 932-2 (BSI, 1999) and tested. The number and mass of test portions varied depending on the test requirements. Fine aggregates are basically sands with most particles passing through a 4mm sieve, while coarse aggregates are particles greater than 4mm (BS EN 12620, BSI, 2002). Traditionally, the coarse aggregates for concrete are graded into sub-size fractions, i.e. 4/10 and 10/20mm.

3.3.1 Classification of aggregates

For recycled aggregates, the proportions of constituent materials were determined in accordance with BS EN 933-11 (BSI, 2009). The test is basically consisting of hand sorting particles from a test portion of recycled aggregates into a list of constituents. The proportion of each constituent was then determined and expressed as a percentage by mass, except for the proportion of floating particles which were expressed as a volume by mass.

3.3.2 Geometrical and Physical properties

Particle size distribution (PSD)

Particle size distribution of an aggregate sample was conducted by sieve analysis as described in BS EN 933-1 (BSI, 2012). At the beginning of the test, the aggregate sample was heated at 110±5 °C for almost 24 hours and the oven dried mass was recorded. The sample was then washed through a 63μm sieve and the material retained on the sieve was oven-dried and weighed again. The mass of fines (material with a particle size smaller than 63μm) was calculated as the difference in the recorded mass of material before and after the washing stage. Next, the remaining test portion was placed on top of a sieve column mounted to a mechanical shaker. After adequate shaking, the sieves were carefully removed one after the other and the mass of material retained was recorded. The cumulative percentage of the original dry mass passing each sieve down to the 63μm sieve was calculated.

Particle Density and water absorption

The density and water absorption of aggregates were determined in accordance with BS EN 1097-6 (BSI, 2013). According to the test method, the representative test sample was initially washed through a particular sieve (63μm and 4mm in case of fine and coarse aggregates, respectively) to remove any fines and then immersed in water at 22±3 °C in a pre-calibrated pycnometer ensuring that all entrapped air was removed. After an immersion period of approximately 24 hours, the pycnometer was filled with water, the top cover was placed, the sides of the vessel were dried and the weight was recorded. The aggregate sample was then removed from water and placed in a tray, where a dry cloth was used to surface-dry the aggregates. The sample was left exposed to air and away from direct sunlight or other source of heat until all visible films of water disappeared before been weighed again. After that, the test portion was place in an oven at 110±5 °C for almost 24 hours and the oven-dried weight was measured. This test procedure allows to determine the apparent, oven-dried and SSD density of the aggregate sample alongside with its water absorption (from oven dried to SSD state).

In case of fine aggregates, the pycnometer was placed in a vacuum chamber to remove the entrapped air and the cone method described in annex F of the standard was used to determine whether the sample reached the SSD state or not.

Particle shape

The assessment of particle shape in terms of a flakiness index was carried out as described in BS EN 933-3 (BSI, 2012) and was only applicable on the coarse aggregates. The flakiness index corresponds to the percentage by weight of particles whose least dimension is less than 0.6 times the mean size dimensions. Similarly to the previous procedures, the test sample was initially oven-dried and the mass was recorded. Two sieving operations were then performed. In the first one, using normal test sieves the sample was separated into various particle size fractions d_i/D_i and the mass of its fraction was weighed. In the second stage, each of the particle size fractions d_i/D_i was sieved again using bar sieves which had parallel slots of width $D_i/2$ and the mass of material passed the bar sieved was recorded this time. The overall flakiness index for the test samples was calculated as the total mass of particles passing the bar sieves expressed as a percentage of the total dry mass of particles tested.

Resistance to fragmentation

The resistance of coarse aggregates to fragmentation was determined in terms of the Los Angeles coefficient as specified in BS EN 1097-2 (BSI, 2010). According to the test procedure, the aggregate sample was initially washed, dried and modified to achieve the grading requirements given by the standard. A portion of 5000g was then placed in a rotating drum and rolled with steel balls for 500 revolutions at a constant speed. After rolling was complete, the quantity of material retained on a 1.6 mm sieve was determined and the Los Angeles coefficient (LA) was calculated from the following equation:

$LA = \frac{5000-m}{50}$, where m is the mass retained on the 1.6 mm sieve, in grams 3-1

Drying Shrinkage

The effect of coarse aggregates on the drying shrinkage of concrete was determined according to BS EN 1367-4 (BSI, 2008). Prior to mixing, the coarse aggregate (4/10 and 10/20 mm) samples were sieved into various size fractions and the mass of material in each fraction was adjusted so that the overall grading curve of the new aggregate sample (4/20 mm) to comply with the grading limits suggested by the standard. The mix proportions were 550g cement, 3300g aggregates (both coarse and fine) and 300g water. The water content of the mix was adjusted for water absorption of the sand while the coarse aggregates were used in SSD condition to prevent any water exchange between the porous aggregates and the mix.

The aggregate sample was mixed with cement and water in a small pan mixer of 10 litre capacity and casted into three prisms (50x50x200 mm) fitted with fixed end studs. The moulds were filled with two approximately equal layers of concrete and sufficient vibration was applied to ensure full compaction. The concrete prisms remained under damp hessians and plastic sheeting for 24 hours before being demoulded and placed into a water tank (20±2 ^{o}C). After 6 days of conditioning in water, the specimens were removed from the water tank (one at a time), the surfaces were wiped with a dry cloth and using a vertical length comparator the initial length was recorded. The specimens were then placed in a ventilated oven at 110±5 ^{o}C for 3 days. Following this, the specimens were placed in a desiccator to cool for approximately 24 hours before the final length measurement. The actual dry length of the prism was also measured using a calliper. The drying shrinkage of concrete attributed to the aggregate was expressed as the average change in length of the 3 prisms, as a percentage of their final dry lengths.

Alkali-Silica Reaction (ASR)

The potential of an aggregate to undergo alkali-silica reaction was evaluated using the screening test described in ASTM C1260 (ASTM, 2014). In this test method, the expansion of mortar specimens immersed in an alkali solution is monitored over a 14 days exposure period and then compared with a set of expansion limits which are indicative of innocuous behaviour or potentially deleterious expansion.

Prior to mixing, the coarse aggregate sample to be tested was crushed into a sand sized material and further processed to achieve the required grading specified in the standard. Mortar pastes were prepared in an orbital 5 litre capacity mixer using 440g of cement, 990g of aggregate and 207g of water. Three gang-moulds (25x25x300 mm) fitted with fixed end studs were filled with mortar and cured for 24 hours, under damp hessian and plastic sheeting, prior to demoulding. The specimens were then placed in a water bath at ≈80^{o}C for a period of 24 hours. Following this, each specimen was removed from the water bath, its initial length was measured (zero reading) with a vertical length comparator and returned in a bath of 1M NaOH at ≈ 80^{o}C. Subsequent comparator readings of the specimens were taken every two days for a period of 14 days after the zero readings. The expansion at each period was expressed as the average change in length of the 3 prisms, as a percentage of their effective gauge length.

The standard states that there is a good agreement with previous literature findings and based on that suggests that expansion of less than 0.10% at 16 days is indicative of innocuous behaviour, of more than 0.20% at 16 days is indicative of potentially deleterious expansions and between 0.10% and 0.20% more information and further testing is required as there is a lot of controversy between lab and field performance. The ASTM C1293 method (also known as the concrete prism test), appears to be the best test method for evaluating deleterious ASR potential in terms of providing the strongest correlation to field performance. However, ASTM C1293 requires a minimum of 1 year testing to evaluate aggregate reactivity. Therefore, ASTM C1260 has a significant practical advantage in its speed.

3.3.3 Chemical

Bulk Oxide Composition – XRF

A Philips PW2424 sequential X-ray Fluorescence Spectrometer with RhKα source was used to determine the bulk oxide composition of the test aggregate and particularly the total chlorides and sulfates contents. The aggregate sample was grinded into powder before placing in a standard pellet mould. The pellet was then compressed under a load of 75kN for 5 minutes and then 150kN for 10 minutes. Certified standard materials were used to calibrate the instrument before testing the samples.

Water soluble sulfates

The water soluble sulfate content of aggregates was determined in accordance with BS EN 1744-1 (BSI, 2012). The procedure to remove water soluble sulfate ions in natural and recycled aggregates is slightly different. In the former situation, the test sample was crushed to pass a 16mm sieve and then transferred to two plastic bottles of 5 litter capacity each. Each bottle contained 2kg of coarse aggregates or 500g of fine aggregates. Water equal to twice the mass of the test specimen was added to the bottle and the contents were mixed by means of a mechanical rolling equipment for at least 24 hours. In case of recycled aggregates, 25g of a test sample previously crushed (if necessary) to pass a 4mm sieve were added to a beaker containing 1000g of hot (60±5 ^{o}C) distilled or demineralized water. The beaker was placed in a hot stirrer plate and the contents were stirred using a magnetic stirrer for approximately 15 minutes. In both cases however, the extract solution was filtered through a dry fine grade filter paper under suction and the determination of water soluble sulfate ions was carried out

using a Metrohm 930 Compact Ion Chromatography (IC) instrument (Figure 3.2a). The results were expressed as percentage by mass of aggregate.

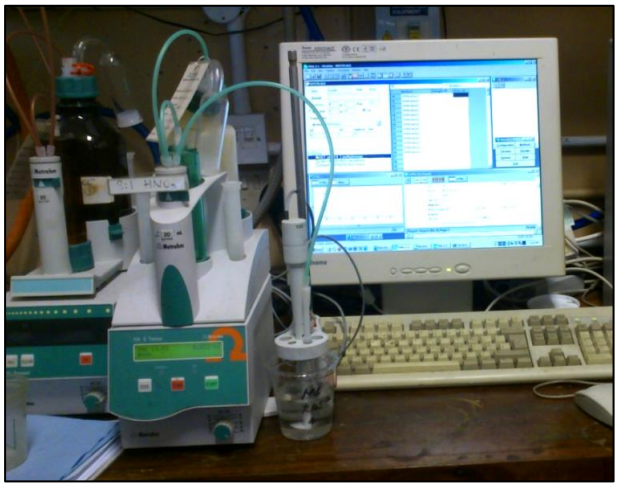
Acid soluble Chlorides

The total (free and bound) chloride content of aggregates was also determined by acid treatment as described in BS EN 14629 (BSI, 2007). Between 1g and 5g of aggregate powder obtained by grinding was digested with 100ml distilled water and 5 mol/l nitric acid. The mixture was heated on a hot plate until boiling and boiled for at least 3 minutes. Stirring was also applied by means of a magnetic stirrer. Following this, the digested solution was filtered thought a dry medium filter paper and the chloride content measured by potentiometric titration (Figure 3.1b). The result was expressed as percentage by weight of aggregate.

Loss on Ignition (LOI)

The LOI of the aggregate samples was measured by burning approximately 1.5g of aggregate powder in a furnace at 975°C (± 25) for 1 hour, as described in BS EN 196-2 (BSI, 2013). The sample was weighed in a crucible, and placed uncovered in the furnace. Following 1 hour of combustion, the crucible was removed and placed in a desiccator. After cooling, the sample was re-weighed and the mass loss calculated. The test was repeated twice for each fly ash and the average taken as the result. The LOI was reported as the loss in mass as a percentage of the original mass of fly ash, after subtracting the moisture content of the sample.

(a)



(b)

Figure 3.2: (a) Ion Chromatography instrument for measuring sulfate ion concentration, (b) Automatic titrator for measuring chloride ion concentration

3.4 **Characterisation tests on CSAB cement**

The relative impact of calcium sulfate in properties of CSAB cement was studied in a performance related approach using variable gypsum additions. Initially, the maximum calcium sulfate content for sufficient ettringite formation was established by the stoichiometric Equation 2-2. This indicated that a 2:1 molar ratio of calcium sulfate to ye'elimite is required to produce ettringite exclusively, which equates to a gypsum demand of 23% w/w replacement of cement. In this regard, gypsum contents of 0, 9, 17 and 23% w/w of CSAB (corresponding to molar ratios of gypsum to ye'elimite of 0, 0.5, 1 and 2) were selected with the aim of controlling the ratio of AFm to AFt hydrates ranging from exclusively AFm to Aft. Other levels of gypsum additions have also been considered in particular tests to optimise results.

Currently, no relevant British or European standards exist for testing CSA cements and hence the British standards for testing cement conforming to BS EN 197-1 (BSI, 2011) were used.

3.4.1 Microstructural analysis

Cement paste samples having gypsum additions of 0, 9, 17 and 123% w/w and a water to binder ratio of 0.5, where 'binder' is used to refer to the combined content of CSAB cement and any added calcium sulfate, were casted and water cured for 7 and 28 days. At each test age, solvent exchange was used to cease the hydration and remove all water in the pores by submerging samples in acetone for 3 days followed by removal of the acetone by oven drying at 30°C, as to minimise drying related artefacts (Galan, et al, 2016). The dried pastes were then ground to a fine powder using a pestle and mortar set. Crystalline hydration products were observed by XRD (Philips PANalytical diffractometer in the Bragg-Brentano geometry) and the extent of hydration was qualitatively measured by peak identification within the XRD patterns of each mixture, using back-loaded samples in order to minimise crystal orientation preference. To determine the effect of mixture proportions on pore characteristics, mercury intrusion porosimetry (MIP) measurements using a Micrometrics Autopore IV 9500 instrument (measuring range of 360 to 0.003 μm) from 0.5 to 60,000 pound per square inch (psia) were also carried out on 7 days hydrated samples.

3.4.2 Setting time

The initial and final setting time was specified in standard consistence pastes using the procedure described in BS EN 196-3 (BSI, 2005). Standard consistence of a cement paste is

defined as the consistence which allow a vicat plunger having 10 mm diameter and 50 mm length (Figure 3.3a) to penetrate to a depth of 33-35 mm from top of the mould.

For CEM I cement pastes this is typically achieved with a w/c ratio of approximately 0.28. Using that value as a starting point and increasing afterward the w/c ratio by 0.2, trial penetration tests were performed on CSAB pastes employing gypsum additions of 0, 1, 2, 4, 6, 9, 17 and 23% w/w to determine the water required for such a consistence. The results were quite variable and it was noted that paste had a tendency to lose plasticity immediately after mixing, possibly due to a false set. This was also apparent to the CSAB pastes having no gypsum addition. In this regard, three w/c ratios equal to 0.3, 0.4 and 0.5 were used to study the effect of variable gypsum contents on setting time of CSAB cement. A retarder compatible with CSA cements was also considered with the aim to regulate the setting.

The setting time was determined in a fresh cement paste sample using an automatic vicat apparatus (Figure 3.3b) and the depth of penetration of a needle into the paste was recorded at pre-specified by the user time intervals. It should be noted that the elapsed period reported is considered from the time that cement first came into contact with water during mixing phase. Figure 3.4 shows a typical setting time graph of a CEM I cement, where point A indicates the initiation of setting phase which corresponds to a penetration depth less than 38 mm, and point B indicates the final setting at which the needle cannot penetrate into the sample more than 0.5 mm.

(a) (b) (c)

Figure 3.3: (a) Typical manual Vicat apparatus for determination of standard consistence mortar, (b) automated Vicat apparatus for determination of setting time, (c) mortar sample tested for setting time.

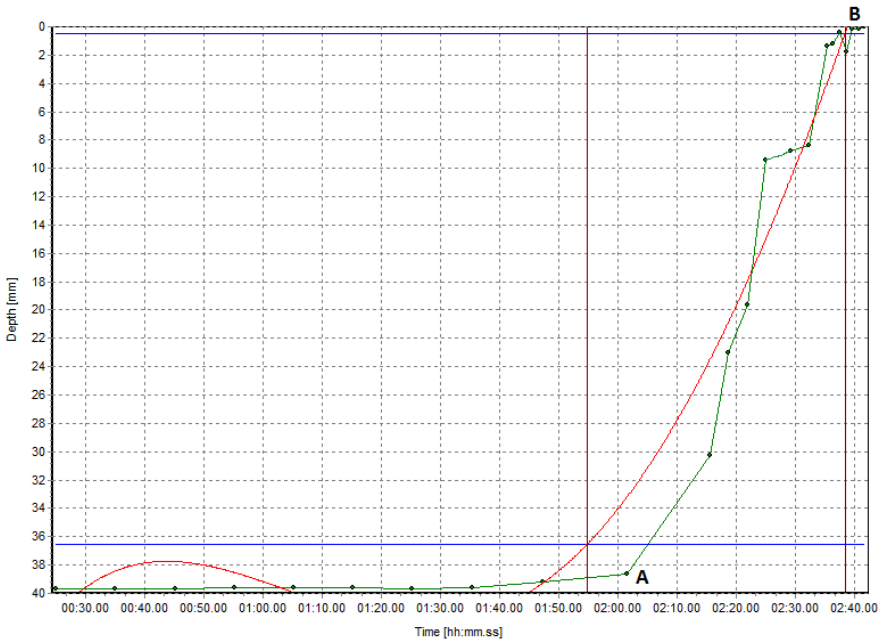


Figure 3.4: Typical setting time graph of PC samples

3.4.3 Alkalinity (pH)

The method used to determine the pH value of CSAB cement is widely known as "pore fluid expression". No standard method exists internationally for measuring this property, however, a common testing protocol was identified in various research papers (Barmeyback and Diamond, 1981; Byfors, et al., 1986, Tritthart, 1989, Alonso, et al., 2012). This mainly involves preparation and curing a cement paste sample which then using a pore pressure device its pore fluid solution is expressed, collected and tested. Factors like w/c ratio, curing conditions, test ages, as well as, the method employed for pH determination were found to vary slightly. Regarding the latter, the pH of the pore solution can be measured directly using a calibrated pH electrode or by indirect titration measurement of OH^- concentration.

Cement paste samples having a water to cement ratio of 0.6 and gypsum content of 0 and 17% w/w were prepared and poured into sealed plastic tubes. The w/c ratio selected here was similar to that used in other studies to overcome the problem of low pore water. The bottles were then stored in a cure chamber with 100% RH at 20 ±2°C for 7 and 28 days (2 samples per testing age). Once curing was completed, the sample placed into the cavity of the cylinder of the pore pressing device (Figure 3.5). Pressure was applied to the piston using a mechanical press and the pore solution is collected in a syringe inserted into the hole of the bottom plate. After collection, the pore fluid was immediately filtered through a 0.45 μm filter and stored in

a sealed plastic container until further analysis. A minimum volume of 5ml was required per testing age. The pH was then determined by OH^- titration.

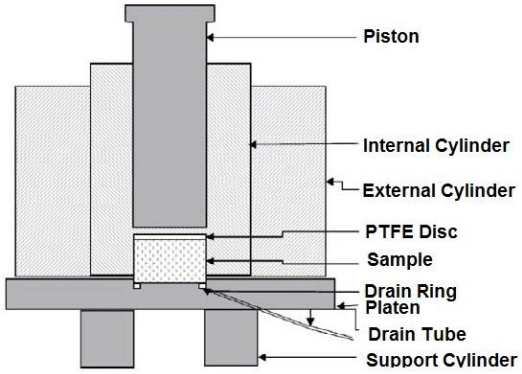


Figure 3.5: Schematic diagram of pore solution expression apparatus (Yonezawa, et al., 1988)

3.4.4 Compressive strength

The compressive strength of CSAB cement was determined in relation to gypsum additions of 0, 1, 2, 3, 5, 9, 17 and 23% w/w following the procedure outlined in BS EN 196-1 (BSI, 2005) for testing CEM cements conforming to BS EN 197-1 (BSI, 2011). The test involves mixing 450g of binder with 225ml water and 1350g sand using an orbital mixer with a stainless steel bowl and placing the resultant mortar mixture in a gang mould (3 No. 40x40x160 mm) and water curing at ~20°C for 28 days. After 28 days of conditioning, the three mortar prisms were split to produce 6 specimens for compressive strength testing. The compressive strength was taken as an average of 6 results. As CSAB cement belongs to the family of rapid hardening cements, additional samples were casted, cured and tested at the age of 3 and 7 days to monitor strength development.

3.4.5 Potential expansion of mortars exposed to sulfates

ASTM C452 (ASTM, 2015) describes a method for determining the potential sulfate resistance of Portland cements only. No other relevant procedure was identified in the literature for non-Portland cements and therefore the method was used to establish the expansion potential of CSAB mortars containing gypsum additions of 0, 1, 2, 3, 5, 9, 17 and 23% by weight of cement.

The test involves mixing 400g of binder (cement and gypsum) with 194ml water and 1100g sand using an orbital mixer with a stainless steel bowl and placing the resultant mortar mixture in a gang mold (3 No. 25x25x300mm) with fixed studs at the ends. After a curing period of 22 to 23 hours under wet hessians the test specimens were removed from molds and

placed in a water tank at 23±2°C. The length was initially measured at approximately 30 minutes after they have been placed in the water tank and then again at 14 days. The results were expressed as a percentage of the original sample length and compared with that of a reference mortar containing CEM I and gypsum in such proportions that the mixture has a total sulfur trioxide (SO_3) content of 7.0 % by mass of binder.

To further monitor the expansion behavior of CSAB cement, subsequent length measurements were also taken at the age of 28, 56 and 84 112, 140 and 168 days.

3.4.6 Chloride ingress

The resistance of CSAB cement to chloride ingress was measured on 28 days water-cured mortar mixtures. The mixes were prepared with a water to binder ratio of 0.5 (for continuity with the prior paste studies), a sand to binder ratio of 2.75 and were having gypsum additions of 0, 9, 17 and 23% w/w. Two widely used test protocols, NT Build 492 (Nordtest, 1999) and NT Build 443 (Nordtest, 1995), were independently used. The NT Build 492 (nordtest, 1999) migration technique is a rapid method of assessing the permeability of concrete to chloride ions by using an electrical charge to accelerate the movement of ions through the test sample. This test method has also been used for assessing the chloride resistance of concrete and therefore more details are given in section 3.6.4. The NT Build 443 (Nordtest, 1995) chloride diffusion technique differs as uses an increased concentration gradient to expedite chloride ingress.

Two disc samples (100 mm diameter, 50 mm thick) were coated with an epoxy membrane such that a single cross-sectional face remained exposed when submerged in a prepared sodium chloride solution (168 g/l) at 20±2 °C for 35 days. An elongated test duration of 6 months was chosen over the prescribed 35 days due to the low chloride ingress rates of these test samples. At the end of the immersion period, the chloride diffusion curve of the exposed profile specimens were established by grinding (using a dry process) a series of ten layers at predetermined depths from the exposed surface. The collected powder of each layer was subsequently tested by means of acid digestion, filtration and potentiometric titration, following the procedure described in NT Build 208 (Nordtest, 1996), to determine the chloride content reported as a percentage by mass of dry mortar powder. The initial chloride content was obtained in the same procedure from an unexposed part of mortar. The calculated chloride diffusion coefficient was determined using a non-linear regression analysis by least

squares curve fitting. The first data point (i.e. the surface layer) was discarded and the regression analysis utilised the data set encompassing those values up to a minimum of the chloride content is Ci, the initial chloride content, plus 0.015%.

3.5 **Concrete tests**

3.5.1 Development of concrete mixes

CEM I recycled aggregate concretes

An extensive laboratory testing programme was carried out to assess the effect of Qatari recycled coarse aggregates on fresh, engineering and durability properties of concrete. Four w/c ratios varying from 0.45 to 0.75 were employed in this study to provide a range of practical mixes and strengths. This enabled the measured properties to be compared on an equal strength basis and interpolation to be carried out so that the requirements outlined in Table A.4 and Table A.12 of BS 8500-1 (BSI, 2015) to be met for 50 year design life (50mm nominal cover) and X0, XC1, XC2, XC3/4, XA1, XD1 and XS1 exposure classes, respectively. These exposure environments are deemed to correspond to medium value concretes in which recycled aggregates have most potential.

The free water content was fixed at 185 l/m^3. This was slightly higher compared to other similar studies (typically in the range of 155-175 l/m^3) but selected here to account for crushed aggregates and maintain the same water content throughout all mixes, given the higher water requirement of CSAB cement. A polycarboxylate based superplasticiser was added, as required, to give an S3 consistence class (slump: 100-150mm) in accordance with BS EN 206 (BSI, 2013). The aggregates were proportioned according to the British Research Establishment (BRE) method for designing normal concrete mixes (BRE, 1997). Coarse granite aggregates were replaced by recycled aggregates. The replacement levels were fixed at 30% and 50% (by volume of granite aggregates) for mixes incorporating CDWA and at 15% and 30% v/v for those containing EWA. The latter levels were lower due to high fines content of EWA resulting in a significantly reduced consistence. For low grade concretes (w/c ≥ 0.65) a total replacement of natural aggregates with CDWA was also considered.

Aggregate replacement was carried out in a volumetric approach and not by mass as it is usual to account for the significant lower densities of recycled aggregates. As the resulting

volumetric proportions of total coarse aggregates, fine aggregate, cement, and water remain constant, no significant losses in the consistence of the concrete should be expected with an increase in the proportion of recycled aggregates as long as their inherently greater absorption (as compared with that of NA) is incorporated into the design. The resultant BRE method material proportions were then all properly adjusted (based on its known densities) to maintain the volumetric yield. The concrete mix proportions are given in Table 3.3. As expected, the theoretical density of recycled aggregate concrete decreases with an increase of recycled aggregate content owing to their adhered mortar on the recycled coarse aggregate.

CEM I control concretes

To validate the results of present study against Qatari practices, a few concrete mixes were prepared with dune sand and coarse gabbro or limestone aggregates as reference materials. These were having a w/c ratio of 0.55 and a fixed recycled aggregate content of 50% v/v CDWA or 30% v/v of EWA. The mix proportions of these 'control' mixes are shown in Table 3.4.

CEM I/FA and CEM I/GGBS recycled aggregate concretes

FA and GGBS in binary blend cement combinations with CEM I at mass replacement levels of 40% and 50%, respectively, were also considered for selective concrete mixes from Table 3.3. Two w/c ratios equal to 0.45 and 0.55 and a fixed CDWA content of 30% v/v were selected here (Table 3.4). This allowed the resulting properties to be compared with that of CEM I recycled aggregate concrete mixes on an equal strength basis and conclusions to be drawn on the achieved durability and sustainability.

CSAB recycled aggregate concretes

CSAB recycled aggregate concrete mixes were designed in a similar approach to CEM I mixes. A free water content of 185 l/m^3 and three w/c ratios equal to 0.55, 0.65 and 0.75 were selected to provide a range of strengths comparable with that of CEM I mixes. In accordance with the findings of the CSAB study in Section 4.2, a gypsum addition of 2% by mass of CSAB cement (denoted hereafter as CSABg) was included in all mixes for optimum performance. This was reduced to 1.5% in the mixes containing recycled aggregates to account for the SO_3 content, typically 1% by mass of coarse recycled aggregates. CDWA were only used in these series of mixes at a fixed level of 50% by volume of coarse granite aggregates. The content of all constituent materials was then properly adjusted based on its known densities to maintain the

volumetric yield. In all mixes, a polycarboxylate-based superplasticiser was added as required to achieve the target slump (100-150mm). The concrete mix proportions are given in Table 3.6.

CSAB/FA and CSAB/GGBS concretes

Selective mixes from Table 3.6 were tested for CSAB cement combinations with FA and GGBS. Previous studies (Zivica, 2001; Ioannou, et al., 2014; Guo and Shi, 2016) have suggested optimal portions of pozzolans up to 15% by mass of cement. In this regard, FA and GGBS were used at 15% direct mass replacement of CSABg at w/c ratios of 0.45 and 0.55. The mix proportions are also given in Table 3.6.

Concrete testing programme

The series of tests considered for CEM I-based and CSAB-based concretes are given in Table 3.8 and Table 3.9, respectively, and analysed further in the following sections.

3.5.2 Concrete mix procedure and testing of fresh properties

Concrete mixing was carried out according to BS 1881-125 (BSI, 2013) in a horizontal pan type mixer of 0.035 m^3 nominal capacity. This involved mixing the laboratory-dry aggregates for 30 seconds, adding half of the water and mixing for a further two minutes. The aggregates were then covered and left for eight minutes for water absorption. The cement (or pre-blended cement combinations with FA) was added and mixed for one minute. Finally, the remaining water along with any admixtures were introduced and mixing continued for a further 2 minutes. After mixing, slump and plastic density of concrete were measured as described in BS EN 12350-2 (BSI, 2009) and BS 12350-6 (BSI, 2000) respectively. The concrete was then re-mixed for 30 seconds before casting and then every 15 minutes during the casting period to maintain it homogeneous and workable. Other properties including cohesiveness, segregation, bleeding, compactibility and finishability were also assessed qualitatively based on visual observations.

In case of recycled aggregate mixes, the rest period for aggregates absorption was extended to approximately 15 minutes to ensure that all aggregates have reached the SSD state. The water compensation method was selected in this study because i) allows conventional and recycled aggregate concrete to be produced in a similar way and ii) pre-saturation method was considered as technically and economically not viable under the special environmental conditions in Qatar (i.e. hot-dry climate and shortage of clean water reserves).

Table 3.3: CEM I recycled aggregate concrete mix proportions

Mix Name	w/c ratio	Agg. Replacement level (v/v), %		SP, % of cement content	Constituent proportions, kg/m³										Total
					Water	Cement		Sand	GRA		CDWA		EWA		
		CCA	EWA			CEM I	SCM	4/0	10/4	20/10	10/4	20/10	10/4	20/10	
Cement type: CEM I 52.5N, Reference coarse aggregate: Granite (GRA)															
CEM I GRA	0.45	0	0	0.41	185	410	0	720	360	715	-	-	-	-	2390
CEM I GRA30CDWA		30	-	0.41					250	500	105	205	-	-	2375
CEM I GRA50CDWA		50	-	0.41					180	355	170	345	-	-	2365
CEM I GRA15EWA		-	15	0.50					305	610	-	-	50	105	2385
CEM I GRA30EWA		-	30	0.50[2]					250	500	-	-	100	210	2375
CEM I GRA	0.55	0	0	0.47	185	335	0	780	360	720	-	-	-	-	2380
CEM I GRA30CDWA		30	-	0.47					250	505	100	210	-	-	2365
CEM I GRA50CDWA		50	-	0.47					180	360	170	345	-	-	2355
CEM I GRA15EWA		-	15	0.58					305	610	-	-	50	105	2370
CEM I GRA30EWA		-	30	0.58[2]					250	505	-	-	100	210	2365
CEM I GRA	0.65	0	0	0.45	185	285	0	835	355	710	-	-	-	-	2370
CEM I GRA30CDWA		30	-	0.45					250	495	100	205	-	-	2355
CEM I GRA50CDWA		50	-	0.45					180	355	170	340	-	-	2350
CEM I GRA100CDWA		100	-	0.45					-	-	345	680	-	-	2330
CEM I GRA15EWA		-	15	0.70					300	605	-	-	50	105	2365
CEM I GRA30EWA		-	30	0.70[2]					250	495	-	-	100	210	2360
CEM I GRA	0.75	0	0	0.53	185	245	0	890	350	695	-	-	-	-	2365
CEM I GRA30CDWA		30	-	0.53					245	490	100	200	-	-	2355
CEM I GRA50CDWA		50	-	0.53					175	350	165	335	-	-	2345
CEM I GRA100CDWA		100	-	0.53					-	-	335	670	-	-	2325
CEM I GRA15EWA		-	15	0.70					300	590	-	-	50	100	2360
CEM I GRA30EWA		-	30	0.70[2]					245	485	-	-	100	205	2355

All concrete mix proportions have been adjusted for yield

Table 3.4: Control recycled aggregate concrete mix proportions

Mix Name	w/c ratio	Agg. Replacement level (v/v), %		SP, % of cement content	Constituent proportions, kg/m³										Total
					Water	Cement		Sand[1]	LA / GA		CDWA		EWA		
		CCA	EWA			CEM I	SCM	0/4	4/10	10/20	4/10	10/20	4/10	10/20	
Cement type: CEM I 52.5N, Reference coarse aggregate: UAE Limestone (LA)															
CEM I LA		0	0	0,32					365	725	-	-	-	-	2400
CEM I LA50CDWA	0.55	50	-	0,32	185	335	0	790	180	365	170	340	-	-	2365
CEM I LA30EWA		-	30	0,42					255	510	-	-	100	205	2380
Cement type: CEM I 52.5N, Reference coarse aggregate: UAE Gabbro (GA)															
CEM I GA		0	0	0,27					380	755	-	-	-	-	2470
CEM I GA50CDWA	0.55	50	-	0,27	185	335	0	815	190	380	165	330	-	-	2400
CEM I GA30EWA		-	30	0,35					265	530	-	-	100	200	2430

[1] Local washed sand from Qatar used for all control mixes

All concrete mix proportions have been adjusted for yield

Table 3.5: CEM I/FA and CEM I/GGBS recycled aggregate concrete mix proportions

Mix Name	w/c ratio	Agg. Replacement level (v/v), %		SP, % of cement content	Constituent proportions, kg/m^3										Total
		CCA	EWA		Water	CEM I	FA	Sand	GRA		CDWA		EWA		
								4/0	10/4	20/10	10/4	20/10	10/4	20/10	
Cement type: CIVB-V (60% CEM I 52.5N + 40% FA), Reference coarse aggregate: Granite (GRA)															
CIVB-V GRA	0.45	0	0	0.3	185	245	165	700	350	700	-	-	-	-	2345
CIVB-V GRA30CDWA		30	-	0.3					245	490	100	200	-	-	2330
CIVB-V GRA	0.55	0	0	0.3	185	200	135	765	350	705	-	-	-	-	2340
CIVB-V GRA30CDWA		30	-	0.3					245	495	100	205	-	-	2330
Cement type: CIIIA (50% CEM I 52.5N + 50% GGBS), Reference coarse aggregate: Granite (GRA)															
CIIIA GRA	0.45	0	0	0.4	185	205	205	715	360	710	-	-	-	-	2380
CIIIA GRA30CDWA		30	-	0.4					250	500	100	205	-	-	2365
CIIIA GRA	0.55	0	0	0.4	185	165	165	780	355	720	-	-	-	-	2370
CIIIA GRA30CDWA		30	-	0.4					250	505	100	205	-	-	2355

All concrete mix proportions have been adjusted for yield

Table 3.6: CSAB recycled aggregate concrete mix proportions

Mix Name	w/c ratio	Agg. Replacement level (v/v), %		SP, % of cement content	Constituent proportions, kg/m³									Total
		CCA	EWA		Water	Cement		Sand	GRA		CDWA		EWA	
						CSABg	SCM	4/0	10/4	20/10	10/4	20/10	10/4	20/10	
Cement type: CSA, Reference coarse aggregate: Granite (GRA)															
CSABg GRA	0.55	0	0	0.34	185	335	0	770	355	715	-	-	-	-	2360
CSABg GRA50CDWA		50	-	0.34					175	355	170	345	-	-	2335
CSABg GRA	0.65	0	0	0.37	185	285	0	830	355	700	-	-	-	-	2355
CSABg GRA50CDWA		50	-	0.37					175	350	170	335	-	-	2330
CSABg GRA	0.75	0	0	0.4	185	245	0	880	350	690	-	-	-	-	2350
CSABg GRA50CDWA		50	-	0.4					175	345	165	335	-	-	2330

All concrete mix proportions have been adjusted for yield

Table 3.7: CSAB/FA and CSAB/GGBS concrete mix proportions

Mix Name	w/c ratio	Agg. Replacement level (v/v), %		SP, % of cement content	Constituent proportions, kg/m³									Total
		CCA	EWA		Water	Cement		Sand	GRA		CDWA		EWA	
						CSABg	SCM	4/0	10/4	20/10	10/4	20/10	10/4	20/10	
Cement type: 85% CSA + 15% FA , Reference coarse aggregate: Granite (GRA)															
CSABg/15FA GRA	0.45	0	0	0,30	185	350	60	705	350	700	-	-	-	-	3350
CSABg/15FA GRA	0.55	0	0	0,34	185	285	50	770	350	705	-	-	-	-	2345
Cement type: 85% CSA + 15% GGBS , Reference coarse aggregate: Granite (GRA)															
CSABg/15FA GRA	0.45	0	0	0,30	185	350	60	710	355	705	-	-	-	-	2365
CSABg/15FA GRA	0.55	0	0	0,34	185	285	50	770	355	715	-	-	-	-	2360

All concrete mix proportions have been adjusted for yield

Table 3.8: CEM I – based concrete testing plan

Mix Name	w/c	Fresh properties	Engineering				Durability		
		Consistence / slump retention	Compressive & flexure strength	Drying shrinkage	ISAT	Carbonation	Chloride migration	Chloride diffusion	Sulfate expansion
CEM I GRA	0.45	✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA50CDWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA15EWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA30EWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA	0.55	✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA50CDWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA15EWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA30EWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA	0.65	✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA50CDWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA100CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA15EWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA30EWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA	0.75	✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA50CDWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓
CEM I GRA100CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA15EWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GRA30EWA		✓ / ×	✓	✓	✓	✓	✓	✓	✓

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Table 3.8: CEM I – based concrete testing plan

Mix Name	w/c	Fresh properties	Engineering		Durability				
		Consistence / slump retention	Compressive & flexure strength	Drying shrinkage	ISAT	Carbonation	Chloride migration	Chloride diffusion	Sulfate expansion
CEM I LA	0.55	✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I LA50CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I LA30EWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GA	0.55	✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GA50CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CEM I GA30EWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CIVB-V GRA	0.45	✓ / ×	✓	×	✓	✓	✓	✓	✓
CIVB-V GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CIVB-V GRA	0.55	✓ / ×	✓	×	✓	✓	✓	✓	✓
CIVB-V GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CIIIA GRA	0.45	✓ / ×	✓	×	✓	✓	✓	✓	✓
CIIIA GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓
CIIIA GRA	0.55	✓ / ×	✓	×	✓	✓	✓	✓	✓
CIIIA GRA30CDWA		✓ / ×	✓	×	✓	✓	✓	✓	✓

Table 3.9: CSAB – based concrete testing plan

Mix Name	w/c	Fresh properties	Engineering				Durability		
		Consistence / slump retention	Compressive & flexure strength	Drying shrinkage	ISAT	Carbonation	Chloride migration	Chloride diffusion	Sulfate expansion
CSABg GRA	0.55	✓ / ✓	✓	✓	✓	✓	×	✓	✓
CSABg GRA50CDWA		✓ / ✓	✓	✓	✓	✓	×	✓	✓
CSABg GRA	0.65	✓ / ✓	✓	✓	✓	✓	×	✓	✓
CSABg GRA50CDWA		✓ / ✓	✓	✓	✓	✓	×	✓	✓
CSABg GRA	0.75	✓ / ✓	✓	✓	✓	✓	×	✓	✓
CSABg GRA50CDWA		✓ / ✓	✓	✓	✓	✓	×	✓	✓
CSABg/15FA GRA	0.45	✓ / ✓	✓	×	✓	✓	×	✓	✓
CSABg/15FA GRA	0.55	✓ / ✓	✓	×	✓	✓	×	✓	✓
CSABg/15GGBS GRA	0.45	✓ / ✓	✓	×	✓	✓	×	✓	✓
CSABg/15GGBS GRA	0.55	✓ / ✓	✓	×	✓	✓	×	✓	✓

3.5.3 Concrete curing

All concrete samples were cured in line with BS EN 12390-2 (BSI, 2009). Following a 24 hours curing period under moist air (95% RH), which ensures that concrete can develop early strength without loss of moisture, the samples were then placed in water tanks at 20±2^{0}C until the age of testing. CEM I and CSAB concrete mixes were cured in different water tanks to prevent interaction of any substances leached out from CEM I concretes with ye'elimite of CSAB concretes.

3.5.4 Concrete testing

Concrete Strength

Compressive strength of concrete was measured as described in BS EN 12390-3 (BSI, 2009) on 100mm cubes and involved testing three cubes at each age. These were carried out at specified intervals (typically 3, 7, 28, 56, 90 and 180 days), with and an average of the measurements reported.

Concrete flexural strength was measured in accordance with BS EN 12390-5 (BSI, 2009). Two prismatic specimens (100x100x500mm) were subjected to a bending moment by the application of load through upper and lower rollers, following a two-point loading arrangement. The maximum load sustained was recorded and the flexural strength was calculated and reported as the average of the two measurements.

ISAT

BS 1881-208 (BSI, 1996) describes an indirect test method to evaluate permeability of concrete by measuring its surface absorption characteristics. Two concrete cubes (150x150x150mm) were casted, water-cured for 28 days, dried at 105±5 ^{o}C until constant mass was achieved and then placed in a desiccator until testing. The test consists of the measurement of water flow into the concrete specimen through a known surface area (Figure 3.6). The contact area is defined by a plastic cell sealed onto the surface. After a defined period (e.g. 10, 30 or 60 minutes) of water flow, the tap is closed and the volume flow is obtained by measuring the length of flow along a capillary tube of known dimensions. It is essential to calibrate the capillary tube before testing so that the movement of water along it during 1 min, as read directly from the scale, equals the initial surface absorption in ml/(m^2 s) at a constant head and temperature during the test.

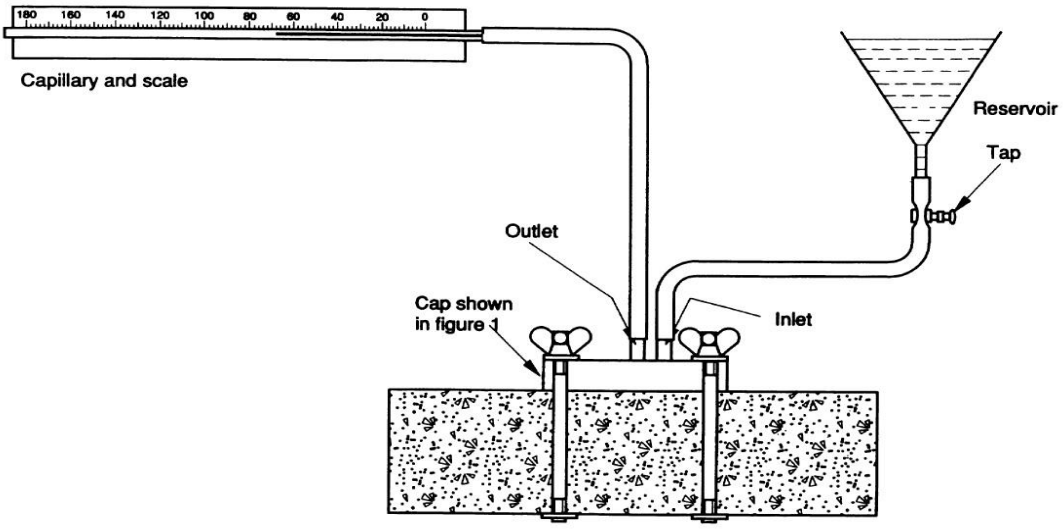


Figure 3.6: Assembly of typical absorption apparatus (BS 1881-208, BSI, 1996)

Drying shrinkage

Drying shrinkage of concrete was determined in accordance with BS ISO 1920-8 (BSI, 2009). Two 75 x 75 x 300 mm prisms fitted with fixed end studs were water-cured for 7 days. Following this, the specimens were removed from the water tank (one at a time) and quickly transferred to an environmental room (22±2 ^{o}C and 55±5 % RH), where all the surfaces were dry wiped with a cloth, the mass was recorded and the initial length was measured using a vertical length comparator. The specimens were then placed in racks inside the room, ensuring a clearance of at least 50mm on all sides. Subsequent measurements were taken in the ages of 14, 21, 28, 56, 84, 112 days as recommended in the standard. For each drying period, the mass loss (as percentage of initial mass) due to water evaporation and the dying shrinkage (expressed in microstrain) were calculated.

Chloride Testing

The transport of chloride ions into concrete (by means of capillary absorption, hydrostatic pressure, and diffusion) is a complicated, multi-mechanistic phenomenon and therefore a variety of testing protocols exist for measuring resistance of concrete to chloride ingress, each offering advantages and disadvantages. In general, these methods can be categorized into two categories: diffusion tests (e.g. AASHTO T259, BS EN 12390-11, NordTest NT Build 443) and migration tests (NordTest NT Build 492, AASHTO T277). As diffusion is the principal chloride transport mechanism from the external environment to concrete and in this regard diffusion tests are preferred over migration techniques even though they require relatively longer

exposure periods to obtain the results. On the other hand, migration techniques offer a rapid means of assessing permeability of concrete to chloride ions by using an electrical charge to accelerate the movement of ions through the concrete. The applicability of migration techniques has significantly increased with time as it appears to correlate well with diffusion tests and field performance. In this study, two widely used test methods, i.e. BS EN 12390-11 (BSI, 2015) and NT Build 492 (Nordtest 1999), where considered for evaluating resistance of concrete to chloride ingress. The majority of testing was carried out with the Nord Test to obtain initial results and then selective mixes were tested in accordance to BS EN 12390-11 (BSI, 2015) to validate the previous findings.

Non-steady state chloride migration tests were carried out following the NT Build 492 method (Nordtest, 1999). A 100 mm Ø, 200 mm long cylinder was cast and water cured for 27 days. Then the top and bottom 20 mm of the cylinder was removed and three 50 mm thick test specimens were cut from the sample. Once the specimens were surface-dried, were vacuum saturated with $Ca(OH)_2$ for 1 hour and kept submerged for a further 23 hours. Thereafter, the test samples were clamped into a rubber sleeve and 300 ml of anolyte solution (0.3M NaOH) poured on the top surface. The encased specimens were then positioned on plastic supports inside a bath containing 12 litres of 10% NaCl solution (catholyte solution) and the anode immersed in the anolyte solution. The cathode was connected to the negative terminal and the anode to the positive terminal of the power supply (Figure 3.7). An electrical potential of 30V DC was initially applied and the temperature of anolyte solution and the initial current were recorded. Based on the initial current, the test voltage and the test duration were selected. At the end of the test, the final current and temperature (of the anolyte) were recorded and the specimens were split and sprayed with 0.1M $AgNO_3$ (silver nitrate) solution. The $AgNO_3$ reacts with the chlorides present and converts to AgCl (silver chloride), a whitish substance (Figure 3.8). In the non-chloride area the silver binds with hydroxides present in concrete, creating a brownish colour. The chloride penetration depth is measured on the basis of the white AgCl precipitation (clearly visible after about 15 minutes) and the non-steady-state migration coefficient determined using the following equation:

$$Dnssm = \frac{0.0239\,(273+T)\,L}{(U-2)t}\ \left(xd - 0.0238\sqrt{\frac{(273+T)\,L\,xd}{U-2}}\right) \qquad 3\text{-}2$$

D_{nssm}, non-steady state migration coefficient , x 10^{-12} m^2/s

U, absolute value of the applied voltage, V

T, average value of the initial and final temperatures in the anolyte solution, °C

L, thickness of specimen, mm

$\mathbf{x_d}$**,** average value of the penetration depths, mm

T, test duration, hours

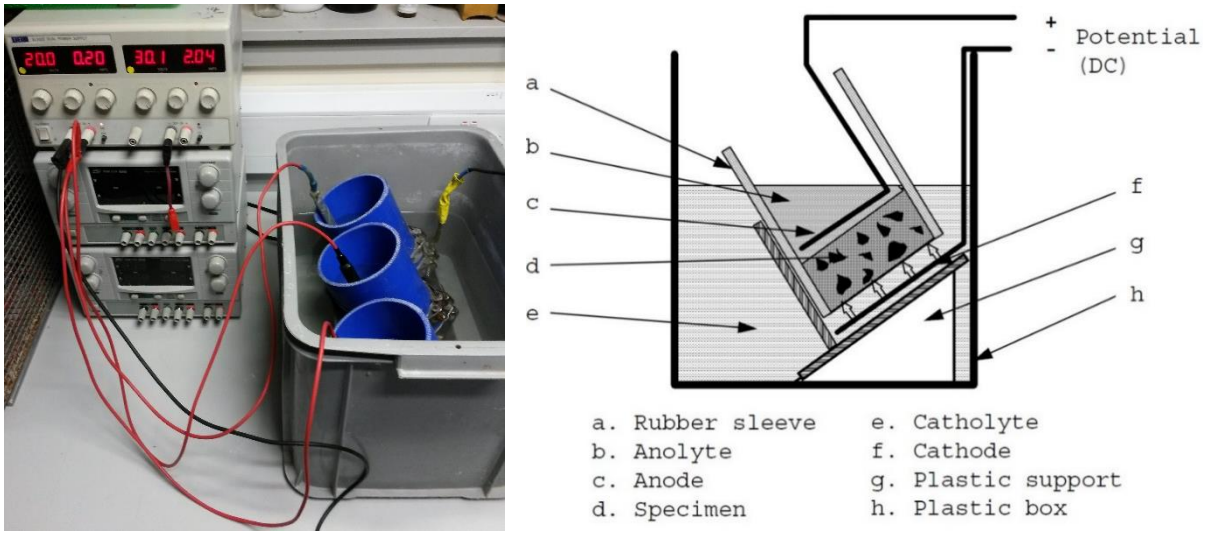


Figure 3.7: Rapid chloride migration test set-up

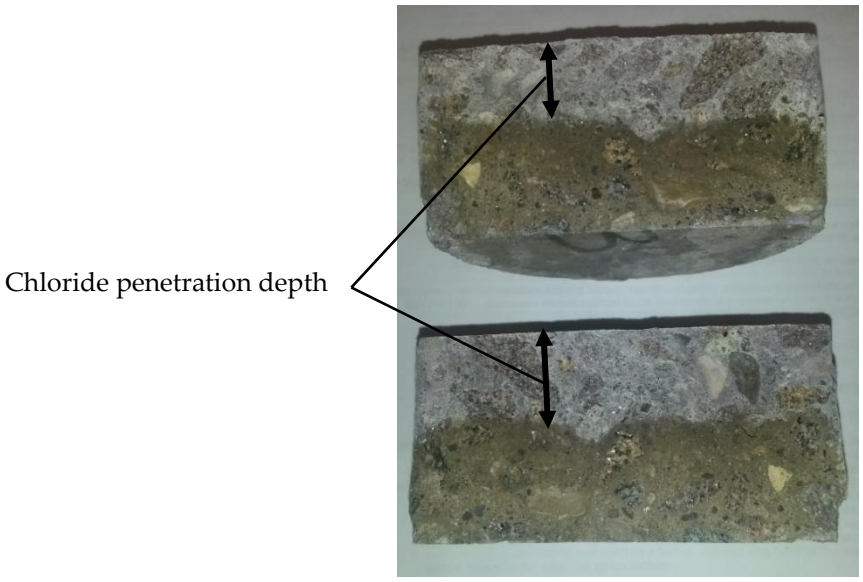


Figure 3.8: Rapid chloride migration test split samples

Natural chloride diffusion under a concentration gradient was carried out to BS EN 12390-11 (BSI, 2015). Two 100mm cubes were cast and cured in water at 20ºC for 28 days. After curing, each cube was cut into two sub-specimens, giving a profile and initial chloride sub-specimen. The initial chloride sup-specimens were used to determine the chloride content of concrete

without exposure and the profile sub-specimens to measure the chloride profile after this had been carried out.

After sawing, all sides, except one (i.e. the cut face), of the profile samples were sealed with wax and then vacuum saturated with distilled water. The cubes were then submerged in a 30 g/l NaCl solution, for 90 days (Figure 3.9). The initial chloride content was determined by grinding between 1 and 5g of powder from the unexposed specimen and digesting this in 5 mol/l nitric acid, as outlined in BS EN 14629 (BSI, 2007). Following this, the digested solution was filtered and the chloride content measured by potentiometric titration.

The chloride diffusion curve of the exposed profile specimen was established by grinding a series of layers (minimum of 8, at set depths) from the exposed surface (unsealed face) of the profile specimen. The grinding depths of each layer were based on recommendations in BS EN 12390-11 (BSI, 2015) and vary depending on the w/c ratio of the mix and whether or not the concrete contains any addition. Table 3.10 and Table 3.11 show the grinding depths for PC and PC/FA concretes, respectively. The chloride content at each of these layers was measured in the same way as the initial chloride content, by acid digestion, filtration and potentiometric titration. The chloride diffusion coefficient was then calculated using non-linear regression analysis by least squares curve fitting. An excel script function programmed by Luping Tang from Chalmers University of Technology, Sweden was used for this analysis. The first data point (i.e. surface layer) is discarded and the regression analysis uses the remaining data points until the chloride content is C_i (initial chloride content) + 0.015% or lower (all remaining data is ignored). The program calculates C_s using the average transport coefficient generated from the data.

Table 3.10: Grinding depth intervals for PC concrete (BS EN 12390-11, BSI, 2015)

	Grinding Depth (mm) in relation to w/c ratio			
	0.40	0.50	0.60	0.70
Layer 1	0-1	0-1	0-1	0-1
Layer 2	1-3	1-3	1-3	1-5
Layer 3	3-5	3-5	3-6	5-10
Layer 4	5-7	5-8	6-10	10-15
Layer 5	7-10	8-12	10-15	15-20
Layer 6	10-13	12-16	15-20	20-25
Layer 7	13-16	16-20	20-25	25-30
Layer 8	16-20	20-25	25-30	30-35

Table 3.11: Grinding depth intervals for fly ash concrete (BS EN 12390-11, BSI, 2015)

	Grinding Depth (mm) in relation to w/c ratio			
	0.40	0.50	0.60	0.70
Layer 1	0-1	0-1	0-1	0-1
Layer 2	1-2	1-3	1-3	1-3
Layer 3	2-3	3-5	3-5	3-6
Layer 4	3-5	5-7	5-8	6-10
Layer 5	5-7	7-10	8-12	10-15
Layer 6	7-9	10-13	12-16	15-20
Layer 7	9-12	13-16	16-20	20-25
Layer 8	12-16	16-20	20-25	25-30

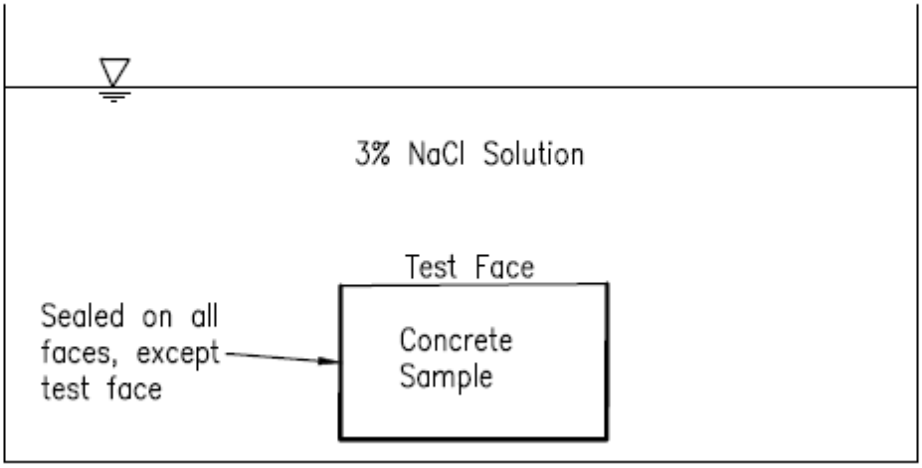
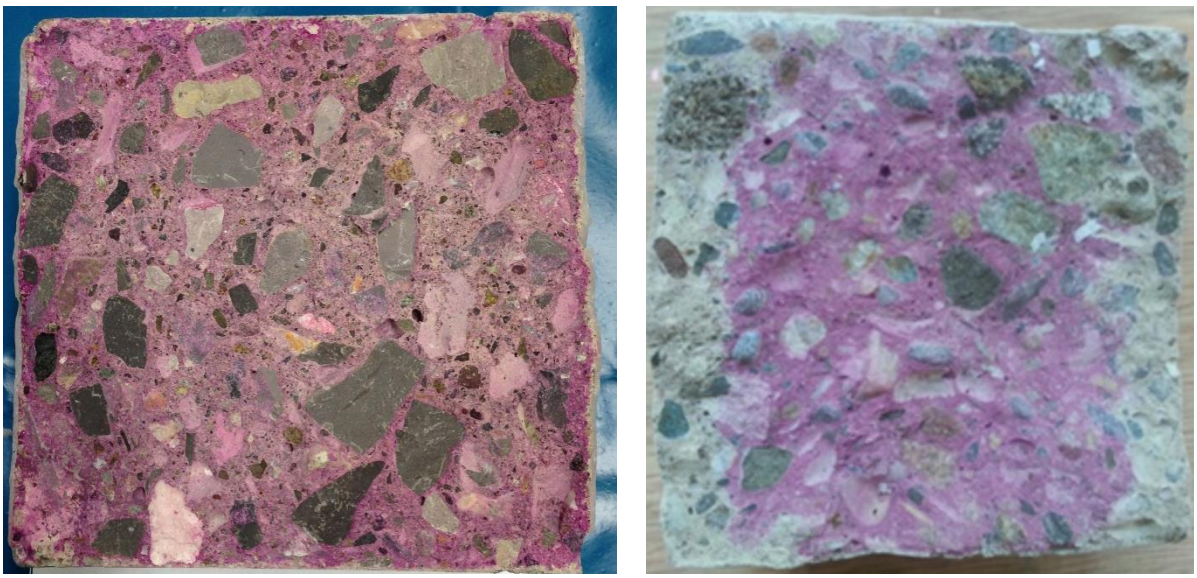


Figure 3.9: Uni-directional diffusion test set-up

Carbonation

Carbonation of concrete is a slow process and under normal exposure conditions (0.035% CO2) an exposure period of 1 to 2 years is required to obtain reasonable test results. For this reason it was decided to test concretes under accelerated conditions (4% CO2). This gives results in a relatively short period of time, enabling the performance of different material combinations to be compared within the timescale of the project. However, the results of the accelerated carbonation tests cannot be extrapolated to a natural-exposure scenario by a simple scaling only related to the ratio of the CO_2 concentrations. In addition to that, the microstructure and transport properties of concrete are changing with time due to the interplay between hydration, pozzolanic (if any) and carbonation reactions, and this may not be reflected under accelerated conditions – thus underestimating the performance of concretes produced with particular cements or cement combinations.

The accelerated carbonation test was performed according to BS 1881-210 (BSI, 2013) and involves water curing two 100x100x500 mm concrete prisms for 28 days, followed by 14 days air curing (20ºC and 55±5% RH) to allow drying. The cast surface and the opposite one were then sealed with wax and the samples placed in storage tanks at 4% CO_2, 20ºC and 55±5% RH for 20 weeks. Although the standard specifies 10 weeks, a longer-term testing was selected to see whether any significant differences would be observed at a later age. At specific periods during the test (1, 2, 4, 6, 8, 10, 12, 14, 16 and 20 weeks) slices were cut from the two prisms and sprayed with phenolphthalein indicator solution. After 12 hours the carbonation depth was measured at 5 points on each edge of the slice and the carbonation depth was calculated as an average of 20 measurements (10 per prism). Examples of carbonation samples sprayed with phenolphthalein indicator solution are shown in Figure 3.10. The colourless region represents the carbonated section of concrete.



(a) (b)

Figure 3.10: Accelerated carbonation slice after (a) 1 week and (b) 10 weeks of exposure

Sulfate Attack

The sulfate resistance of concrete can be assessed by preparing realistic concrete specimens and placing them in conditions which are representative of field conditions. Unless the concretes are of low quality (i.e. high w/c ratio, poorly compacted) several years of exposure are required to obtain meaningful discrimination results between different material combinations. Consequently, there was a need for accelerated test procedures that would

provide discrimination within a timescale of months. CEN/TR 15697 (CEN, 2008) gives a number of options to evaluate concrete performance to sulfate attack and used in this study.

Following the recommendations in CEN/TR 15697 (CEN, 2008), three options were considered for testing the sulfate resistance of concrete samples, full immersion, partial immersion and cyclic wetting and drying in sulfate solution. Of these, full immersion and cyclic testing were selected. In the former scenario the concrete deterioration is caused by the chemical action of sulfate attack while in the latter one the resulting deterioration is due to a combined effect of both the physical action of cyclic crystallisation of the sulfate salts and its chemical action.

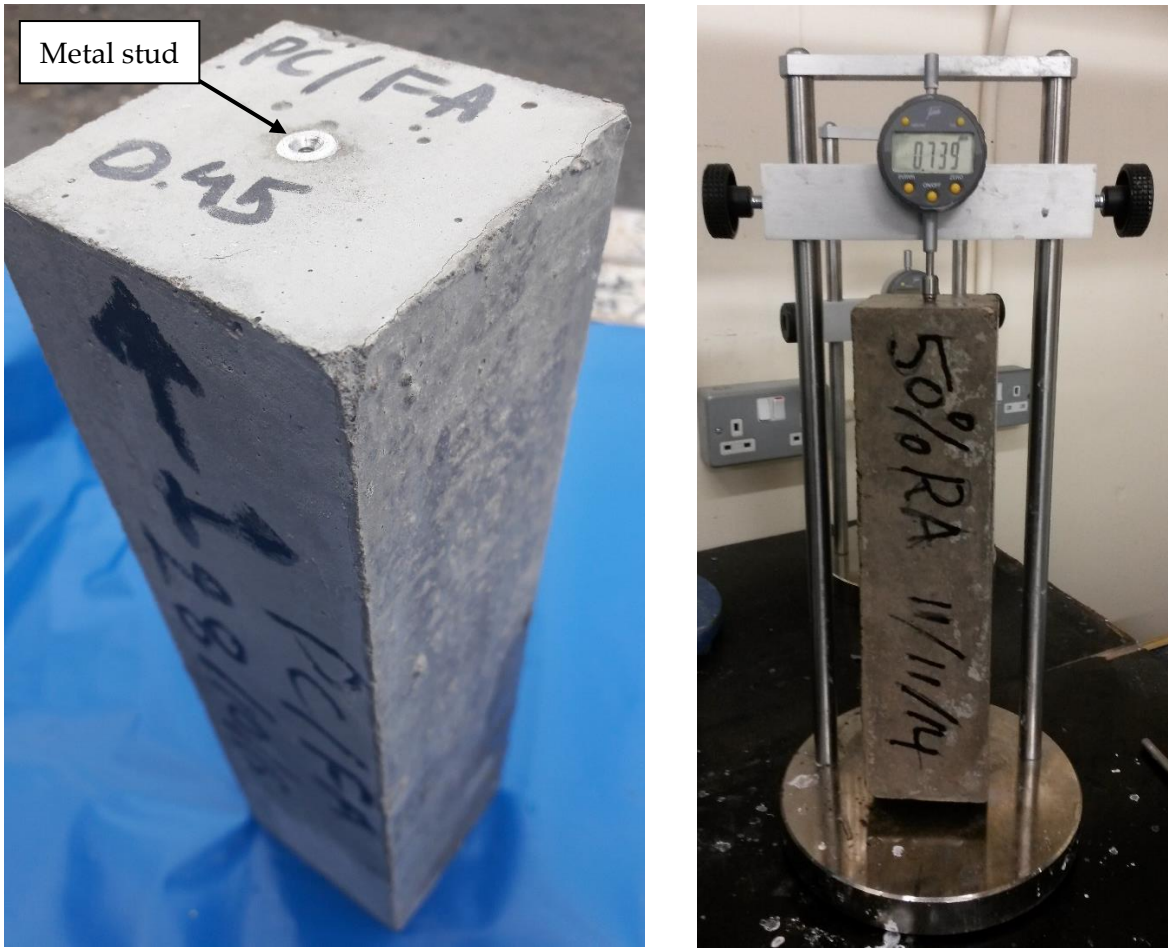
With regard to the type of test solution, two types of sulfates were considered to form the test solution, sodium sulfate and magnesium sulfate. For the same concentration, magnesium sulfate is more aggressive and destructive from sodium sulfate and therefore may mask the variation between test materials. This is because the magnesium sulfate also contributes to deterioration by interacting with lime to form magnesium hydroxide, which effects the pH of the concrete and results in destabilization of CSH and formation of non-cementitious materials (magnesium silicate hydrate) (Rasheeduzzafar, et al., 1994). Conversely, sodium sulfate cause deterioration by cracking through the formation of expansive crystalline compounds (ettringite and gypsum) (Rasheeduzzafar, et al., 1994). Subsequently, the sodium does not influence results and the mechanism of attack is less destructive and may therefore allow differences in performance to be measured more accurately.

In terms of concentration of the test solution, immersion in 14g/l Na_2SO_4 would reflect exposure to a high sulfate soil in the field (BS 8500-1, BSI, 2015). However, due to the nature of sulfate attack this would not necessarily yield measurable results in the timescale of the study. Therefore, the highest concentration recommended by the standard, that of 68 g/l, was chosen as it can produce significant discrimination results after 1 to 2 years (Stephens and Carrasquillo, 2000).

Concrete testing involved curing four 75 x 75 x 300 mm prisms fitted with fixed end studs (Figure 3.11a), for 28 days. Following this, the initial length and mass of the specimens were measured and then two of the them were stored in a tank containing 68 g/l sodium sulfate solution (Na_2SO_4) at 20°C and the other two placed in a tidal tank (18h immersion, 6h drying) under the same conditions. In both cases, the specimens were positioned vertically to ensure

all faces were exposed to the solution, which was replaced at regular intervals (6 months). Change in length and mass of specimens was measured using a length comparator (Figure 3.11b) and a precision scale, and the results reported as a percentage with respect to the original sample length and mass, respectively.

The performance of the concretes was evaluated by determining the change in length and mass at regular intervals over a period of 60 weeks. The BRE Digest 330 (BRE, 2004) classes concretes which expand less than 0.1% during AAR testing as non-expansive. In the context of sulfate attack, others (Samarai, 1976; Patzias, 1991) have also considered 0.1% as the limit above which significant degradation could occur.



(a) (b)

Figure 3.11: (a) 75x75x300mm concrete prism and (b) length comparator

CHAPTER 4: **CHARACTERISTICS OF MATERIALS**

This Chapter gives the characterisation results of the recycled aggregates and CSAB cement. The properties described below were determined following test methods (outlined in Section 3.5 and 3.6) described in British/European standards, or following established techniques identified in the literature.

4.1 **Characteristics range of recycled aggregates**

Twenty six samples of recycled aggregates comprising aggregates from processed construction and demolition wastes (CDWA) and aggregates from processed excavated wastes (EWA) were obtained in three size fractions i.e. 0/4, 4/10, 10/20 from Qatar and tested accordingly. The properties of natural aggregates typically used in Gulf Region and UK and obtained to benchmark the performance of recycled aggregates were also included here.

Table 4.1 gives the details and coding of all the aggregate samples used in the study. In the coding designation the letters refer to the aggregate type. The former number indicates the maximum aggregate size and the number following that refers to a particular sample.

4.1.1 Composition of recycled aggregates

Coarse recycled aggregates are classified depending on their composition. In BS EN 12620 (BSI, 2002), type CCA is required to have a minimum R_{cu} content of 90%, as these are deemed to be the materials with the best mechanical properties, compared to the generic recycled aggregate class where no requirement is applied. Table 4.2 and Table 4.3 give the composition of recycled aggregates and it can be seen that all aggregates can be classed as CCA based on BS EN 12620 (BSI, 2002) and QCS5-2 (QS, 2014) or as Type A in accordance with BS EN 206 (BSI, 2013).

The CDWA samples were mainly comprised from unbound and hydraulically bound aggregates (Ru) and to a less extent of concrete and mortar particles (Rc). However, the Rc content in the CDWA aggregates was found to increase in the last batches, probably due to different processing techniques been implemented and/or structural elements been crushed. Similarly, natural limestone stones and unbound aggregates (Ru) constituted the majority of coarse EWA particles. No shell content was found in any of the aggregate samples, while the presence of other constituent materials (e.g. bituminous materials, glass particles, plastics, metals, bricks and tiles) was minor.

Table 4.1: Summary of aggregate test samples

Received date	Sample description (source)	Sample code	Quantity, kg
March 2014	CDWA - 20mm (Qatar)	CDWA20-1	100
	CDWA - 10mm (Qatar)	CDWA10-1	100
	CDWA - 4mm (Qatar)	CDWA4-1	100
	EWA-20mm (Qatar)	EWA20-1	100
	EWA-10mm (Qatar)	EWA10-1	100
	EWA-4mm (Qatar)	EWA4-1	100
	Gabbro-20mm (UAE)	GA20-1	100
	Gabbro-10mm (UAE)	GA10-1	100
	Limestone-20mm (UAE)	LA20-1	100
	Limestone-10mm (UAE)	LA10-1	100
	Washed sand-4mm (Qatar)	WS4-1	100
June 2014	CDWA - 20mm (Qatar)	CDWA20-2	100
	CDWA - 10mm (Qatar)	CDWA10-2	100
	CDWA - 4mm (Qatar)	CDWA4-2	100
October 2014	CDWA - 20mm (Qatar)	CDWA20-3	250
	CDWA - 10mm (Qatar)	CDWA10-3	250
November 2014	EWA-20mm (Qatar)	EWA20-2	150
	EWA-10mm (Qatar)	EWA10-2	150
	EWA-4mm (Qatar)	EWA4-2	150
	Gabbro-20mm (UAE)	GA20-2	150
	Gabbro-10mm (UAE)	GA10-2	150
	Limestone-20mm (UAE)	LA20-2	150
	Limestone-10mm (UAE)	LA10-2	150
	Washed sand-4mm (Qatar)	WS4-2	150
December 2014	Granite-20mm (UK)	GRA20-1	3000
	Granite-10mm (UK)	GRA10-1	3000
	Sand-4mm (UK)	S4-1	4000
January 2015	CDWA - 20mm (Qatar)	CDWA20-4	500
	CDWA - 20mm (Qatar)	CDWA20-5	500
	CDWA - 10mm (Qatar)	CDWA10-4	500
	CDWA - 10mm (Qatar)	CDWA10-5	500
	CDWA - 4mm (Qatar)	CDWA4-3	500
	CDWA - 4mm (Qatar)	CDWA4-4	500
	EWA-20mm (Qatar)	EWA20-3	500
	EWA-20mm (Qatar)	EWA20-4	500
	EWA-10mm (Qatar)	EWA10-3	500
	EWA-10mm (Qatar)	EWA10-4	500
	EWA-4mm (Qatar)	EWA4-3	500
	EWA-4mm (Qatar)	EWA4-4	500
	Washed sand-4mm (Qatar)	WS4-3	500
	Washed sand-4mm (Qatar)	WS4-4	500

Table 4.2: Composition of coarse (10/20) recycled aggregates

Constituent, %	CDWA20					EWA20			
	1	2	3	4	5	1	2	3	4
Rc	7.0	11.0	27.5	28.5	36.0	1.5	2.5	3.0	3.0
Ru	92.0	88.0	71.5	71.0	63.0	98.0	96.5	95.5	96.5
Rb	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
Ra	0.5	0.5	1.0	< 0.5	1.0	0.5	1.0	1.0	0.5
Rg	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
F_L	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
X	< 0.5	< 0.5	< 0.5	0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
Rcu = Rc + Ru	99.0	99.0	99.0	99.5	99.0	99.5	99.0	98.5	99.5
Shell content	none					none			

Table 4.3: Composition of coarse (4/10) recycled aggregates

Constituent, %	CDWA10					EWA10			
	1	2	3	4	5	1	2	3	4
Rc	18.0	14.5	21.0	19.5	19.0	3.0	1.5	4.0	5.0
Ru	81.5	85	78.5	80.0	80.0	96.0	97.5	94.5	93.0
Rb	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
Ra	0.5	< 0.5	0.5	< 0.5	1.0	1.0	1.0	1.5	2.0
Rg	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
F_L	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
X	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5	< 0.5
Rcu = Rc + Ru	99.5	99.5	99.5	99.5	99.0	99.0	99.0	98.5	98.0
Shell content	none					none			

4.1.2 Grading and fines content

Coarse and fine aggregate should be graded in such a way as to minimise the voidage. The particle size distribution of recycled and natural aggregates along with their fines content is shown in Table 4.4 to Table 4.6. The general grading requirements set in BS EN 12620 (BSI, 2002) for coarse and fine aggregates (QCS5-2 conforms to the same grading limits) are shown in Table 4.7 and compared with that of recycled aggregates in Figure 4.1 and Figure 4.2.

In general, most coarse and fine CDWA and EWA samples were found not to fully conform to the grading requirements. For instance, in CCA20-2&3 samples, the material passing the 10 mm sieve size accounted for 53% w/w and 33% w/w, respectively, when it should have only been in the range of 0% to 20% w/w. Similarly, sample EWA10-2 contained an excess of material above the size of 10mm and obviously above 2 mm, as the material passing the 0.063

mm was more than 5% w/w. Samples EWA10-3&4 have also not conformed to the limit of 0-5% w/w set for material finer than 2 mm for the same reason. The gradation of all EWA4 samples was found within the specified limits but that of CDWA4 appeared to be quite coarser and did not meet the criterion of at least 90% w/w of material passing the 4mm.

Table 4.4: Particle size distribution of coarse (10/20) aggregates

Size, mm	CDWA20					EWA20				GA20		LA20		GRA20
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
	Accumulative Percent passing sieve, % by mass													
28	100	100	100	100	100	100	100	100	100	100	100	100	100	100
20	83	99	97	98	99	98	98	97	98	95	91	96	94	97
16	48	87	71	59	63	67	74	66	75	55	43	68	63	49
14	24	78	60	36	43	51	61	52	63	33	38	53	47	23
10	5.0	53	33	13	9.0	15	14	14	22	5.0	2.0	14	9.0	4.0
6.3	3.0	14	8.0	3.0	2.0	2.0	5.0	5.0	8.0	1.0	1.0	3.0	2.0	2.0
4.0	2.0	1.2	1.5	2.3	1.6	1.0	4.3	3.8	5.9	0.3	0.5	1.2	0.9	1.4
0.063	1.1	0.5	0.5	1.4	1.3	0.6	2.6	2.4	3.2	0.2	0.4	0.9	0.7	0.5
	Fines category (BS EN 12620, 2002)													
	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	f_4	f_4	f_4	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$

Table 4.5: Particle size distribution of coarse (4/10) aggregates

Size, mm	CDWA10					EWA10				GA10		LA10		GRA10
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
	Accumulative Percent passing sieve, % by mass													
20	100	100	100	100	100	100	100	100	100	100	100	100	100	100
14	99	100	98	100	100	100	94	100	100	100	100	100	100	100
10	89	95	97	97	97	99	65	85	90	96	86	92	94	85
8.0	51	57	67	62	66	64	38	55	61	65	57	63	60	48
6.3	16	27	40	32	39	32	19	28	35	40	29	35	34	10
4.0	2.0	5.6	12.5	6.4	9.0	4.0	14	16	18	6.8	5.7	8.9	10.1	4.1
0.063	1.2	1.3	2.0	1.7	1.9	2.3	6.9	9.5	10.5	0.7	1.5	1.2	1.8	0.7
	Fines category (BS EN 12620, 2002)													
	$f_{1.5}$	$f_{1.5}$	f_4	f_4	f_4	f_4	$f_{6.9}$	$f_{9.5}$	$f_{10.5}$	$f_{1.5}$	$f_{1.5}$	$f_{1.5}$	f_4	$f_{1.5}$

Table 4.6: Particle size distribution of fine (0/4) aggregates

Size, mm	CDWA4				EWA4				WS4				S4
	1	2	3	4	1	2	3	4	1	2	3	4	1
	Accumulative Percent passing sieve, % by mass												
8.0	100	100	100	100	100	100	100	100	100	100	100	100	100
6.3	98	97	91	94	98	97	100	100	99	99	99	99	99
4.0	67	65	76	82	91	90	95	95	98	97	97	97	96
2.0	56	56	62	68	76	70	81.5	82.5	93	91	91	90	81
1.0	50	48	50	55	56	45	69	70	83	79	79	79	62
0.5	32	35	31	33	45	25	57	58	48	48	48	47	42
0.25	19	21	12	13	34	14	38	43	11.5	12	12	13	17
0.125	6.8	6.5	2.7	3.5	11	5.0	11	13.5	1.0	0.9	0.8	0.6	3.5
0.063	2.0	2.8	0.9	1.3	4.4	2.4	5.1	6.7	0.1	0.2	0.1	0.1	0.9
	Fines category (BS EN 12620, 2002)												
	f_3	f_3	f_3	f_3	f_{10}	f_3	f_{10}	f_{10}	f_3	f_3	f_3	f_3	f_3

Table 4.7: Grading requirements for coarse and fine aggregates (BS EN 12620, 2002)

Aggregate	Size	Percentage passing by mass					Category
		2D	1.4D	D	d	d/2	
Coarse	D/d ≤ 2 or D ≤ 11.2 mm	100	98-100	85-99	0-20	0-5	G_c 85/20
		100	98-100	80-99	0-20	0-5	G_c 80/20
Fine	D ≤ 4 mm and d=0	100	95-100	85-99	-	-	G_F 85

D: upper sieve size, d: lower sieve size

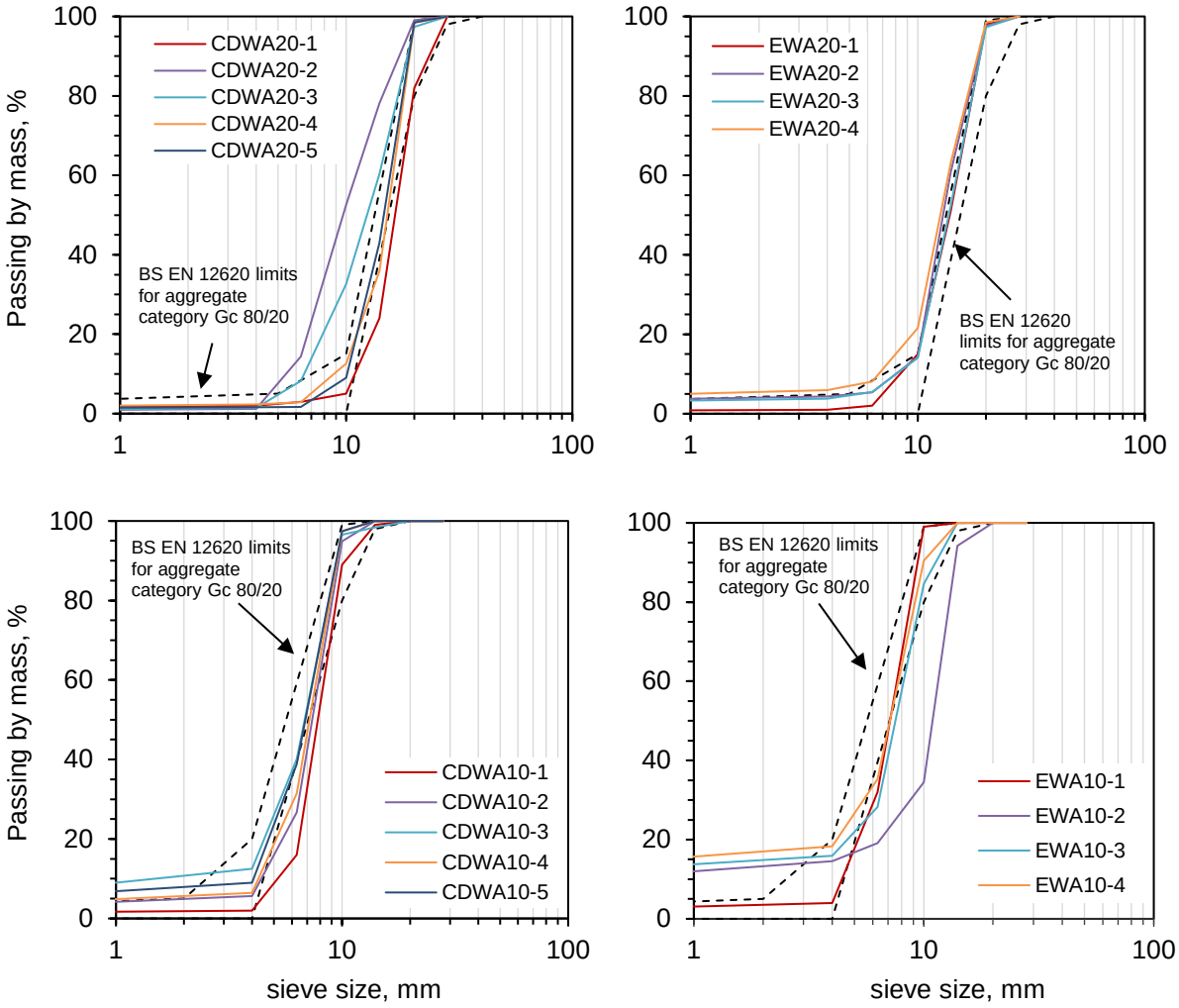


Figure 4.1: Particle size distribution of coarse (10/20 & 4/10) recycled aggregates

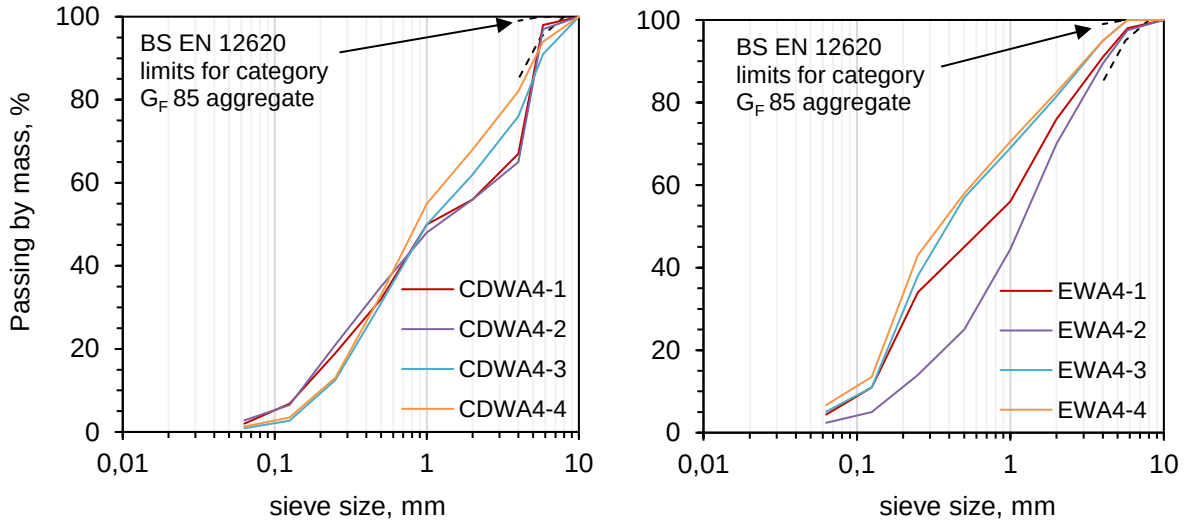


Figure 4.2: Particle size distribution of fine (0/4) recycled aggregates

In case of a particular recycled aggregate type, the particle size distribution curves of individual samples were found to vary significantly, with inconsistence being higher in case of CDWA20, EWA10 and EWA4 samples. This strengthen the speculation that recycled aggregates have been processed differently during the 1-year collection period.

The standards also apply restrictions on the fines content of aggregates as an increased present of fines can lead to consistence and shrinkage problems when used in concrete. BS EN 12620 (BSI, 2002) and BS EN 8500-2 (BSI, 2014) limit this content to 4% w/w for coarse aggregates, whereas QCS5-2 (QS, 2014) applies a limit of 2% w/w for coarse aggregates and 3% w/w for fine aggregates. The fines content of CDWA samples were all below 2% w/w and conforming to both aforementioned limits. The EWA20 samples were conforming only to British standards as the fines content was less than 4% w/w, while the 10 mm and 4 mm size fractions appeared to have very high fines contents (≥7% w/w) and didn't meet the criteria set in both specifications. A possible explanation for the high presence of fines might be the lack of proper screening or dust being wind swept over the site whilst the material was stockpiled.

Natural aggregates, both coarse and fine, showed a consistent gradation, low fines content and all conformed to British and Qatari construction standards. The insignificant fines content found in washed sand was expected as the material has been screened and washed thoroughly to remove most contaminants.

4.1.3 Particle density and water absorption

BS EN 12620 (BSI, 2002) and QCS5-2 (QS, 2014) specify suitable aggregates for normal weight concrete applications, including concrete in conformity with BS EN 206 (BSI, 2013) and concrete used in roads and other pavements and precast concrete products, as those having an oven-dried density greater than 2000 kg/m^3. While BS EN 12620 (BSI, 2002) does not apply any limit on the water absorption of aggregates, Qatari specifications restrict water absorption values of natural aggregates to 2% w/w for coarse and 2.3% w/w for fine, and in case of recycled aggregates (all size fractions) to 3% w/w and 4% w/w for structural and non-structural concrete, respectively.

Table 4.8 to Table 4.10 give the results for particle density (apparent, oven-dried and saturated surface dry (SSD) and water absorption of all tested aggregates. Both natural and recycled

aggregates were classed as normal weight aggregates, since their oven-dried particle density was higher than 2000 kg/m^3.

It is also well documented that recycled aggregates typically display lower densities and higher absorption values compared to natural aggregates. This is mainly attributed to the porous nature of cement paste attached to the aggregates. (Poon and Chan, 2006; Juan and Gutiérrez, 2009; Tam and Tam, 2009). Indeed, the density of all recycled aggregates found to be lower and the water absorption to be higher from those of imported aggregates and washed sand. The extend of variation was found to differ depending on the size and type of recycled aggregate and therefore an average value (AVE), a standard deviation (STD) and a coefficient of variation (COV) are provided in Table 4.11 to Table 4.13 to allow conclusions to be made. It can be seen that coarse CDWA samples have lower density and higher water absorption values from EWA, but their properties were less varied. The significant variation observed in water absorption values of EWA samples indicated a substantial heterogeneity of local limestones in terms of porosity, which was in line with other research findings (Fourniadis, 2010; Al-Ansary and Iyengal, 2013). It is also noteworthy that all recycled aggregates have absorption values higher than 3% w/w and in most cases higher than 4% w/w exceeding the limits set by current Qatari standards for structural and non-structural concrete, respectively, making these aggregates not suitable for use in any type of concrete applications. The substantial high absorption characteristics of CDW4 would be prohibitive for their implementation in construction even for the UK industry where BS EN 12620 (BSI, 2002) allows the use of fine recycled aggregates in concrete if adequate performance is proven in a case by case basis.

The water absorption of EWA4 samples couldn't be measured with accuracy (was in the range of 15 % to 20% w/w) due to difficulties judging whether the material has reached the SSD state or not, and thus have not been reported. Other researcher (You, 2009; Rodrigues, et al., 2013) have also expressed concerns on the adopted by the standards cone method to determine the SSD condition of angular, rough or porous fine aggregates as these materials present cohesiveness and binding properties and do not slump in the same way as natural sands. Alternative techniques have been proposed but either involve the use of expensive equipment or their accuracy and reproducibility are still questioned.

While the existence of cement paste could explain the lower density and higher water absorption properties of CDWA, this was not the case for EWA. As highlighted in the

literature, excavation wastes are arising from earthworks and mainly comprised of limestone fragments. These local limestone fragments are known to be weak and porous compared to high quality limestone sources exist elsewhere (e.g. UAE or Oman). If we also consider that most structures across the Gulf region were built in the previous century using local limestone aggregates, then a combination of a poor quality parent aggregate with residual cement paste would justify the even lower properties of CDWA over EWA samples.

Researchers (Poon and Chan, 2006; Juan and Gutiérrez, 2009; Tam and Tam, 2009) have demonstrated that the attached cement paste content correlates well both with saturated surface-dry density and water absorption of aggregates. In particular, as the cement paste content increases, the density of aggregates decreases and the water absorption rises due to increased porosity. The typical practise to quantify the cement paste content of aggregates by acid treatment was not applicable in this study as both types of recycled aggregates contained limestone particles which are known to react with acid. However, a strong relationship was found between the SSD density and water absorption of all aggregates and illustrated in Figure 4.3. The figure also shows that the material properties are changing with fineness, i.e. finer aggregate fractions display lower densities and higher absorption values. This is in line with other observations (Chandra, 2004; Katz, 2004; Poon and Chan, 2006) and can be explained by the relatively higher amounts of crushed cement paste and fine particles accumulating in the finer aggregate fractions after various processing stages.

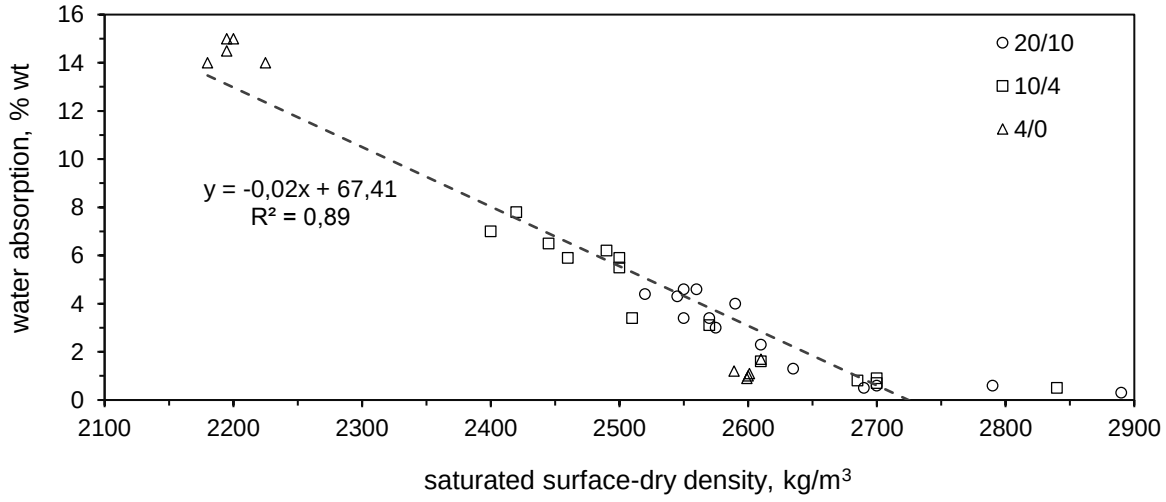


Figure 4.3: Relationship between the SSD density and water absorption value

Table 4.8: Density and water absorption of coarse (10/20) aggregates

Property	CDWA20					EWA20				GA20		LA20		GRA10
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
Apparent particle density, kg/m³	2750	2730	2740	2720	2690	2710	2710	2700	2690	2810	2900	2720	2705	2685
Oven-dried particle density, kg/m³	2440	2500	2440	2435	2410	2550	2485	2500	2465	2770	2880	2690	2680	2600
SSD particle density, kg/m³	2560	2590	2550	2545	2520	2610	2570	2575	2550	2790	2890	2700	2690	2635
Water absorption, % wt	4.6	4.0	4.6	4.3	4.4	2.3	3.4	3.0	3.4	0.6	0.3	0.6	0.5	1.3

Table 4.9: Density and water absorption of coarse (4/10) aggregates

Property	CDWA10					EWA10				GA10		LA10		GRA10
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
Apparent particle density, kg/m³	2730	2660	2715	2735	2720	2700	2705	2685	2690	2740	2870	2720	2705	2675
Oven-dried particle density, kg/m³	2360	2230	2240	2345	2370	2490	2395	2320	2295	2680	2830	2680	2665	2570
SSD particle density, kg/m³	2500	2400	2420	2490	2500	2570	2510	2460	2445	2700	2840	2700	2685	2610
Water absorption, % wt	5.9	7.0	7.8	6.2	5.5	3.1	3.4	5.9	6.5	0.9	0.5	0.7	0.8	1.6

Table 4.10: Density and water absorption of fine (0/4) aggregates

	CDWA4				EWA4				WS4				S4
	1	2	3	4	1	2	3	4	1	2	3	4	1
Apparent particle density, kg/m³	2615	2605	2660	2650	2450	2585	2615	2595	2641	2644	2644	2631	2705
Oven-dried particle density, kg/m³	1920	1910	1960	1915					2574	2568	2570	2560	2555
SSD particle density, kg/m³	2195	2180	2225	2195			*		2601	2599	2600	2589	2610
Water absorption, % wt	14.5	14	14	15					1.1	0.9	1.0	1.2	1.7

* couldn't be measured

Table 4.11: Density and water absorption variation of coarse (10/20) aggregates

Property	CDWA20			EWA20			GA20			LA20			GRA20
	Mean	STD	COV	Mean	STD	COV	Mean	STD	COV	Mean	STD	COV	
Apparent particle density, kg/m³	2725	23	0.8	2705	10	0.4	2855	64	2.2	2715	11	0.4	2685
Oven-dried particle density, kg/m³	2445	33	1.4	2500	36	1.5	2825	78	2.8	2685	7	0.3	2600
SSD particle density, kg/m³	2555	25	1.0	2575	25	1.0	2840	71	2.5	2695	7	0.3	2635
Water absorption, % wt	4.4	0.2	5.7	3.0	0.5	17.2	0.5	0.2	47.1	0.6	0.1	12.9	1.3

Table 4.12: Density and water absorption variation of coarse (4/10) aggregates

Property	CDWA10			EWA10			GA10			LA10			GRA10
	Mean	STD	COV	Mean	STD	COV	Mean	STD	COV	Mean	STD	COV	
Apparent particle density, kg/m³	2710	30	1.1	2695	9	0.3	2805	92	3.3	2715	11	0.4	2675
Oven-dried particle density, kg/m³	2310	68	3.0	2375	88	3.7	2755	106	3.8	2675	11	0.4	2570
SSD particle density, kg/m³	2460	48	2.0	2495	56	2.3	2770	99	3.6	2695	11	0.4	2610
Water absorption, % wt	6.5	0.9	14.2	4.7	1.7	36.5	0.7	0.3	40.4	0.8	0.1	9.4	1.6

Table 4.13: Density and water absorption variation of coarse (0/4) aggregates

Property	CDWA4			EWA4			WS4			S4
	Mean	STD	COV	Mean	STD	COV	Mean	STD	COV	
Apparent particle density, kg/m³	2635	27	1.0	2695	9	0.3	2640	6	0.2	2705
Oven-dried particle density, kg/m³	1925	23	1.2				2570	6	0.2	2555
SSD particle density, kg/m³	2200	19	0.9		*		2600	6	0.2	2610
Water absorption, % wt	14.4	0.5	3.3				1.1	0.1	12.3	1.7

* couldn't be measured

4.1.4 Particle shape

An equidimensional shaped coarse aggregate particle is preferred as flaky particles tend to reduce consistence of concrete and cause problems with bleed water and air voids forming underneath particles. The mass of flaky particles expressed as a percentage of sample mass is known as the flakiness index and the results are given in Table 4.14 and Table 4.15. BS EN 12620 (BSI, 2002) recommends a maximum FI value of 50%, while QCS5-2 (QS, 2014) limits that to 35%. Figure 4.4 and Figure 4.5 show the particle shape and angularity of the coarse fraction of typical CDWA, EWA, GA and LA samples.

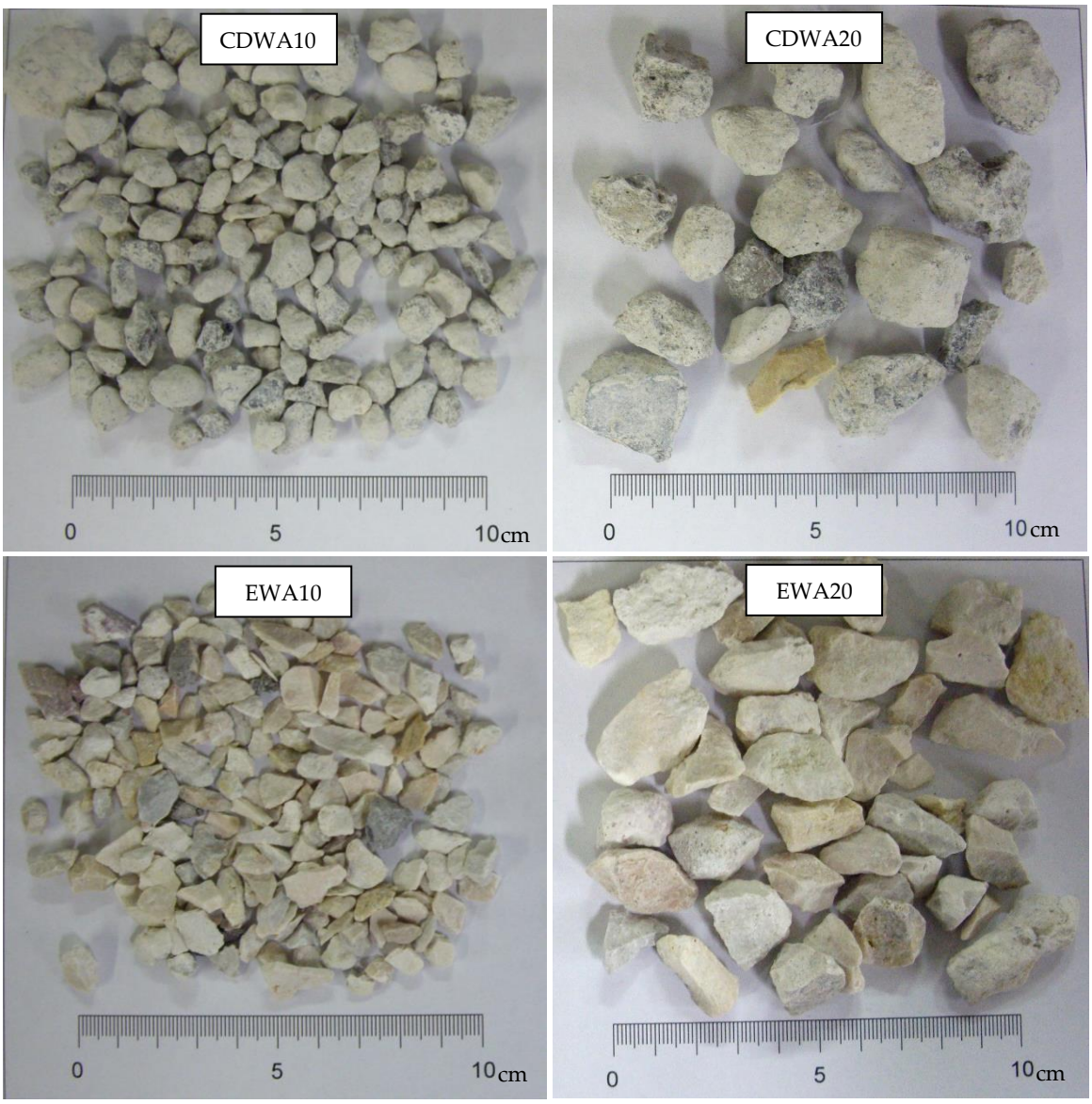


Figure 4.4. Particle shape of typical recycled aggregate samples

Figure 4.5: Particle shape of typical UAE natural aggregate samples

All recycled aggregate samples had a FI value of less than 35% and hence were conforming to the limits set in both specifications. For the same size fraction CDWA and EWA showed a relatively small variation in FI values, with the former aggregate type also displaying noticeably lower values. Low FI values correspond to a more spherical particle shape and as it can be seen in Figure 4.4 and Figure 4.5. CDWA appeared to be more rounded compared to EWA.

The aggregate flakiness index was also found to reduce with size fraction. It has been mentioned in the literature that the number of processing stages and the types of crushers used have a significant influence on the resulting shape of aggregates. Although the processing

methods used on site were unknown, it could be assumed that 10/20 size fraction aggregates resulted after a primary crushing stage where only a jaw crusher might has been used, whereas a cone or impact crusher might has been used in a secondary crushing stage for the production of 4/10 size aggregates.

As expected, all natural aggregates had FI values within the limits mentioned above. The high variability noticed between the FI values of gabbro or limestone aggregate samples of equal size fraction strengthen the speculation that the materials have been produced in different quarries and/or under different processing methods.

4.1.5 Resistance to Fragmentation - Loss by Loss Angeles Abrasion test

The resistance to fragmentation of the coarse (10/20) aggregate fraction of all aggregates is given in Table 4.14 in terms of a Loss Angeles (LA) abrasion index value. Higher values indicate a reduced resistance to fragmentation. BS EN 12620 (BSI, 2002) suggests a maximum LA value of 50 for construction aggregates, whereas BS 8500-2 (BSI, 2015) recommends 40 and suggest in other cases further testing with concrete trials to establish the suitability of the aggregates with respect to concrete compressive and shear strength. Qatari construction standards reduce further this limit to 30. All aggregates were found to comply with the strict limit of 30. Recycled aggregates performed satisfactory and displayed LA values similar to those of imported limestone aggregates. The substantially low LA value of gabbro aggregates compared to the others was expected due to the high density of the material. A good relationship has also been established between the oven-dried density of aggregates and its corresponding resistance to fragmentation.

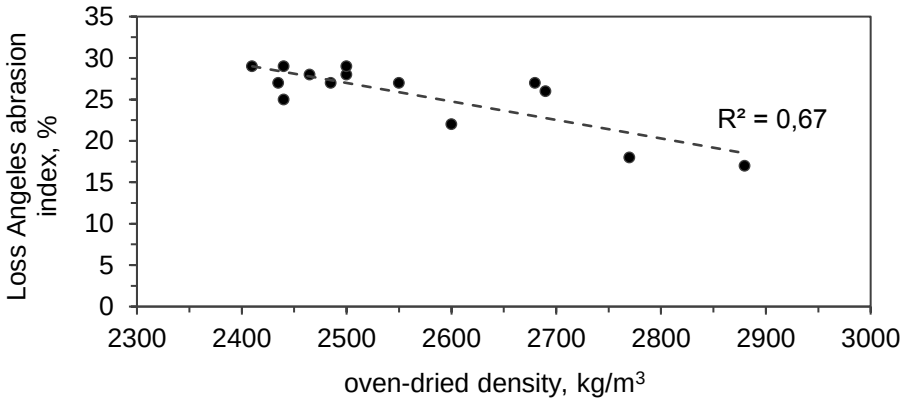


Figure 4.6: Relationship between oven-dried density and Loss Angeles abrasion Index of aggregates.

4.1.6 Alkali Silica Reaction (ASR)

Alkali-aggregate reaction (AAR) is an expansive reaction within hardened concrete and occurs between the alkali hydroxides, originated mainly from the cement, and reactive silica in certain types of susceptible aggregate (Dent Glasser and Kataoka, 1981; Dyer, 2014). Damaging AAR is rare on a world scale and has not been reported as a significant cause of concrete deterioration in the Arabian Peninsula (The Concrete Society and CIRIA, 2002); however the possibility of deleterious chemical reactivity must be evaluated.

Alkali-aggregate reaction has two forms: Alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). From these two the former reaction is of more concern because aggregates containing reactive silica constituents (e.g. opal, chalcedony, tridymite, acidic volcanic glass and microcrystalline or cryptocrystalline quartz) are more common. French and Poole (1976) and Sims and Poole (2017) have reported the existence of potentially reactive aggregates materials in the region and Fearson (1996) has also noted expansive alkali-silica reactions in some concretes in southern Oman. Testing aggregates for reactivity potential is difficult, because considerable experience is needed to interpret the results and because some methods require up to a year or more to produce reliable results. Table 4.14 and Table 4.15 show the results obtained from an accelerated screening test method used to detect the potential of an aggregate to undergo ASR. Prisms made both with recycled and natural coarse aggregates have shown expansion values significantly below the limit of 0.1% set by the ASTM C1260 (ASTM, 2014) and therefore no risk of ASR exist.

4.1.7 Drying shrinkage

Compared to the potential contraction of cement paste under a loss of pore water, shrinkage of construction aggregates is generally considered insignificant. However, the high porosity and associated water demand of recycled aggregates makes the material more prone to volume changes. BS EN 1260 (BSI, 2002) and QCS5-2 (QS, 2014) limit the shrinkage originated from aggregates to 0.075% and as Table 4.16 shows all aggregates (10 and 20 mm combined for the test) conform to this limit. As also expected, mixes containing CCA or EWA exhibited higher shrinkage due to its increased porosity.

4.1.8 Chemical properties

Oxide composition

BS EN 206 (BSI, 2013) place a limit on the chloride content of concrete of between 0.2%-1.0% by mass of cement based on the sum of the contributions from all constituents. Chloride contributions from recycled aggregates for use in this calculation were measured by an acid-soluble test (BS EN 1744-5, BSI, 2006) that provides a worst-case value and probably overestimates the availability of chlorides – thus providing a margin of safety. QCS5-2 (QS, 2014) limit is more conservative and restrict chloride content of coarse and fine aggregates to 0.03% and 0.06%, respectively.

According to BS EN 206 (BSI, 2013), the maximum acid-soluble sulfate content of recycled aggregate must be determined on a case-by-case basis and the water-soluble sulfate content not to exceed the 0.7% by mass of aggregates. BS EN 12620 (BSI, 2002) limits the acid-soluble sulfate content of all aggregates types to 0.8% by mass and the water-soluble sulfate content of recycled aggregates to 0.2%. Similarly, QCS5-2 (QS, 2014) place a limit of 1% on the total sulfate content of CCA, but don't provide any requirements for the water-soluble one.

Table 4.17 to Table 4.19 give the bulk oxide analysis for all aggregates received. CDWA samples are mainly comprised from CaO, SiO_2, Al_2O_3, Fe_2O_3 and MgO oxides whilst the EWA are mostly made up of CaO and MgO (with a smaller amount of SiO_2). The total and acid-soluble chloride contents of coarse aggregates were all found to be relatively low and within the requirements of BS EN 206 (BSI, 2013) but couldn't met that of acid-soluble chlorides in QCS5-2 (QS, 2014); thus not suitable for use in reinforced concrete in Qatar. It is noteworthy that even gabbro aggregates which have been used extensively in construction exceeded slightly the limit of 0.03%. The majority of recycled aggregate samples were also displayed values for total sulfur below the limit of 1% set by BS EN 12620 (BSI, 2002). It was mainly the 4/10 mm size fraction and particularly the EWA samples that found with extremely high total sulfate contents. Acid-soluble sulfate contents of 20mm recycled aggregates generally conformed to the requirements of both specifications, but those of 4/10 mm were mostly above the even more tolerant limit of 1% included in QCS5-2 (QS, 2014). The water-soluble sulfate content of CDWA samples was mainly below 0.2% but all EWA samples failed to meet this limit. The presence of sulfates in natural aggregates was minor.

Similarly to the coarse fraction, the chloride content (total and acid-soluble) of all fine recycled aggregates was low and conform to BS EN 12620 (BSI, 2002) but not with QCS5-2 (QS, 2014). Only washed sand found to have less acid-soluble chlorides than the 0.06% limit set by the Qatari standards. The total, acid-soluble and water-soluble sulfate contents of fine recycled aggregate samples were all significantly higher and didn't comply with any requirements set in both specifications. Again was notable the high sulfate content present in washed sand, which did not conform to the Qatari specifications, i.e. for typical fine aggregates used in normal concrete the limit for acid-soluble sulfates is set to 0.4% by mass.

The loss-on-ignition values were found to range from 20% to 25% for CDWA and between 40% and 45% for EWA samples. The high values of EWA samples could be associated with decomposition of contained limestone.

Mineralogy

The mineralogical composition was determined using X-ray diffraction techniques. Figure 4.7 shows typical XRD traces of CDWA and EWA samples. Belite, portlandite, gypsum, ettringite, calcite, dolomite, quartz, albite and kaolinite were found in CDWA samples, which are typically present in recycled concrete recovered from a disposal site and also reported in other studies (Limbachiya, et al., 2007). Excavation wastes contained mainly quartz, calcite, dolomite, albite, gypsum and clay minerals, such as kaolinite and palygorskite. These phases are common in aggregates in general, with the presence of dolomite and gypsum being typical in the region. From the components observed, especially the sulfate containing ones (gypsum and ettringite) are of particular importance from a recycling point of view. This is due to volume stability issues occurring if excess sulfate forms expansive reaction products (e.g. ettringite) in the presence of calcium and aluminium sources in hydrating conditions. Although Portland cement does contain added gypsum to avoid flash-set i.e. diverting the hydration reactions from the formation of calcium aluminate hydrates to that of calcium aluminate sulfate hydrates, more than about 3% gypsum equivalent of sulfate may pose risks and should be closely controlled. From this aspect, the application of calcium sulfoaluminate cements, investigated in the remit of this study, bears a particular advantage on top of its green credentials (lower embodied energy and concomitant carbon footprint), namely, it is more tolerant to an excess of sulfate.

Table 4.14. Mechanical properties of coarse (20/10) aggregates

Property	CCA20					EWA20				GA20		LA20		GRA20
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
Flakiness Index, %	12.2	12.0	13.7	15.1	14.7	26.1	20.0	23.1	25.8	16.2	10.9	25.5	23.5	29.9
BS EN 12620 category	FL_{15}	FL_{15}	FL_{15}	FL_{20}	FL_{15}	FL_{30}	FL_{20}	FL_{25}	FL_{30}	FL_{20}	FL_{15}	FL_{30}	FL_{25}	FL_{30}
LA Value	25	28	29	27	29	27	27	29	28	18	17	26	27	22
BS EN 12620 category	LA_{25}	LA_{30}	LA_{30}	LA_{30}	LA_{30}	LA_{30}	LA_{30}	LA_{30}	LA_{30}	LA_{20}	LA_{20}	LA_{30}	LA_{30}	LA_{25}
ASR Expansion, %	0.019	0.026	0.027	0.024	0.021	0.063	0.054	0.056	0.050	0.012	0.010	0.016	0.017	0.014

Table 4.15. Mechanical properties of coarse (10/4) aggregates

Property	CCA10					EWA10				GA10		LA10		GRA10
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
Flakiness Index, %	4.8	5.0	7.1	8.5	8.5	8.1	12.0	12.3	12.2	19.1	13.9	23.9	16.1	22.4
BS EN 12620 category	FL_{10}	FL_{10}	FL_{10}	FL_{10}	FL_{10}	FL_{10}	FL_{15}	FL_{15}	FL_{15}	FL_{20}	FL_{15}	FL_{25}	FL_{20}	FL_{25}
LA Value BS EN 12620 category	na													
ASR Expansion, %	0.021	0.036	0.029	0.027	0.020	0.038	0.048	0.035	0.034	0.013	0.016	0.020	0.019	0.015

na = not applicable

Table 4.16. Drying shrinkage of coarse aggregates

Property	CCA					EWA				GA		LA		GRA
	1	2	3	4	5	1	2	3	4	1	2	1	2	1
Drying shrinkage, %	0.069	0.066	0.072	0.067	0.059	0.047	0.062	0.046	0.048	0.055	0.049	0.030	0.032	0.045

Table 4.17. Bulk oxide composition and chemical properties of of coarse (10/20) aggregate

Oxide,%	CCA20					EWA20				GA20		LA20		GRA20
by mass	1	2	3	4	5	1	2	3	4	1	2	1	2	1
CaO	39.6	47.4	42.3	43.5	44.7	50.8	49.8	51.2	50.6	3.2	10.0	68.9	78.3	nt
SiO_2	29.0	21.7	24.9	22.2	22.5	6.1	9.9	8.6	8.7	41.0	45.1	5.0	2.2	nt
Al_2O_3	5.5	4.3	4.7	4.5	4.1	0.9	1.8	1.1	1.2	1.2	11.6	0.3	0.7	nt
$Fe2O_3$	4.0	3.5	3.8	3.4	3.4	0.6	1.2	0.8	0.7	9.0	7.9	1.2	0.5	nt
MgO	7.9	6.7	8.2	9.4	8.2	17.9	15.3	15.8	16.0	39.6	24.7	8.3	1.1	nt
MnO	0.1	0.1	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.1	<0.1	-	nt
TiO_2	0.3	0.2	0.2	0.2	0.2	<0.1	0.1	0.1	0.1	<0.1	0.2	-	0.1	nt
K_2O	0.4	0.4	0.4	0.3	0.4	0.1	0.1	0.1	0.1	<0.1	<0.1	<0.1	0.2	nt
Na_2O	0.5	0.4	0.5	0.5	0.5	0.2	0.2	0.2	0.2	0.1	0.6	0.1	0.1	nt
P_2O_5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	nt
Cl	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	<0.1	<0.1	nt
SO_3	0.8	1.4	1.0	0.9	1.0	0.6	1.1	0.5	0.8	0.2	0.2	0.1	0.2	nt
Cl*	0.05	0.06	0.09	0.07	0.06	0.07	0.07	0.07	0.07	0.04	0.04	0.03	0.03	nt
SO_3*	0.72	1.11	0.84	0.80	0.88	0.21	0.92	0.43	0.74	0.18	0.17	0.08	0.19	nt
SO_3**	0.21	0.22	0.20	0.13	0.13	0.27	0.36	0.26	0.29	<0.1	<0.1	<0.1	<0.1	nt
LOI	19.3	24.1	20.2	24.7	23.6	43.7	40.1	42.2	38.5	10.1	5.3	42.5	41.7	nt

* Acid-soluble, ** Water-soluble, nt = not tested

Table 4.18. Bulk oxide composition and chemical properties of of coarse (4/10) aggregate

Oxide,%	CCA10					EWA10				GA10		LA10		GRA10
by mass	1	2	3	4	5	1	2	3	4	1	2	1	2	1
CaO	47.0	44.3	44.0	40.4	42.2	52.4	46.9	46.2	46.1	1.5	7.7	80.6	76.2	nt
SiO_2	24.4	24.5	24.7	26.4	24.5	7.0	12.6	13.6	13.6	42.6	44.7	1.5	3.8	nt
Al_2O_3	3.9	4.0	4.2	4.3	3.9	1.0	2.1	2.4	2.5	1.0	9.1	0.3	1.2	nt
$Fe2O_3$	2.9	3.5	3.5	3.7	3.9	0.6	1.1	1.4	1.3	8.7	7.6	0.3	0.5	nt
MgO	5.5	7.4	7.0	7.8	8.6	16.5	15.0	14.9	14.7	40.8	29.0	2.1	1.7	nt
MnO	0.1	0.1	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.1	<0.1	-	nt
TiO_2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.2	0.2	<0.1	0.1	-	0.1	nt
K_2O	0.5	0.4	0.4	0.5	0.4	0.1	0.2	0.2	0.2	<0.1	<0.1	0.1	0.3	nt
Na_2O	0.4	0.5	0.4	0.5	0.5	0.2	0.3	0.3	0.3	0.1	0.6	0.1	0.1	nt
P_2O_5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	nt
Cl	0.1	0.1	0.1	0.2	0.1	0.1	0.2	0.2	0.2	<0.1	0.1	<0.1	<0.1	nt
SO_3	1.0	1.2	1.0	1.3	1.1	0.5	2.1	2.8	2.5	0.2	0.2	0.1	0.3	nt
Cl*	0.05	0.06	0.05	0.08	0.07	0.07	0.10	0.12	0.11	0.04	0.04	0.02	0.02	nt
SO_3*	0.91	1.09	0.94	1.22	1.01	1.43	1.99	2.68	2.39	0.16	0.18	0.07	0.25	nt
SO_3**	0.26	0.21	0.18	0.18	0.16	0.39	0.92	1.66	1.15	<0.1	<0.1	<0.1	<0.1	nt
LOI	23.6	21.6	22.4	21.6	22.8	42.2	39	42.3	40.4	11.5	6.8	40.7	42.2	nt

* Acid-soluble, $ Water-soluble, nt = not tested

Table 4.19. Bulk oxide composition and chemical properties of fine (0/4) aggregate

Oxide, % by mass	CCA4				EWA4				WS4				S4
	1	2	3	4	1	2	3	4	1	2	3	4	1
CaO	38.5	42.5	45.2	38.0	45.6	43.2	38.9	40.0	12.2	14.6	13.4	13.0	nt
SiO_2	33.6	28.8	27.2	34.4	16.0	19.3	22.7	23.0	64.5	60.9	62.7	63.6	nt
Al_2O_3	4.7	4.7	4.7	4.4	2.7	3.1	4.2	4.0	4.7	5.0	4.9	4.8	nt
$Fe2O_3$	2.6	2.6	2.7	2.3	1.3	1.4	1.9	1.8	0.8	1.0	0.9	0.8	nt
MgO	4.7	4.9	4.3	4.6	13.7	11.9	10.2	10.5	1.1	1.4	1.2	1.1	nt
MnO	0.1	0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	nt
TiO_2	0.2	0.2	0.3	0.2	0.1	0.2	0.3	0.2	0.1	0.2	0.1	0.1	nt
K_2O	0.8	0.7	0.7	0.8	0.2	0.3	0.5	0.5	1.8	1.7	1.7	1.8	nt
Na_2O	0.5	0.5	0.5	0.5	0.3	0.4	0.4	0.4	1.4	1.5	1.4	1.4	nt
P_2O_5	< 0.1	< 0.1	0.1	0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	nt
Cl	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.1	< 0.1	0.1	0.1	nt
SO_3	2.1	2.5	3.5	2.5	3.2	5.4	9.7	8.1	1.7	1.5	1.6	1.7	nt
Cl*	0.16	0.10	0.07	0.08	0.11	0.15	0.17	0.16	0.04	0.03	0.04	0.04	nt
SO_3*	2.01	2.27	3.36	2.33	3.08	5.25	9.59	7.98	1.56	1.42	1.48	1.55	nt
SO_3**	0.27	0.32	0.28	0.22	0.98	1.21	1.64	1.77	0.12	0.1	0.12	0.13	nt
LOI	14.7	16.6	16.8	16.9	42.2	34	30.2	30.5	5.6	5.2	6.1	5.8	nt

* Acid-soluble, ** Water-soluble, nt = not tested

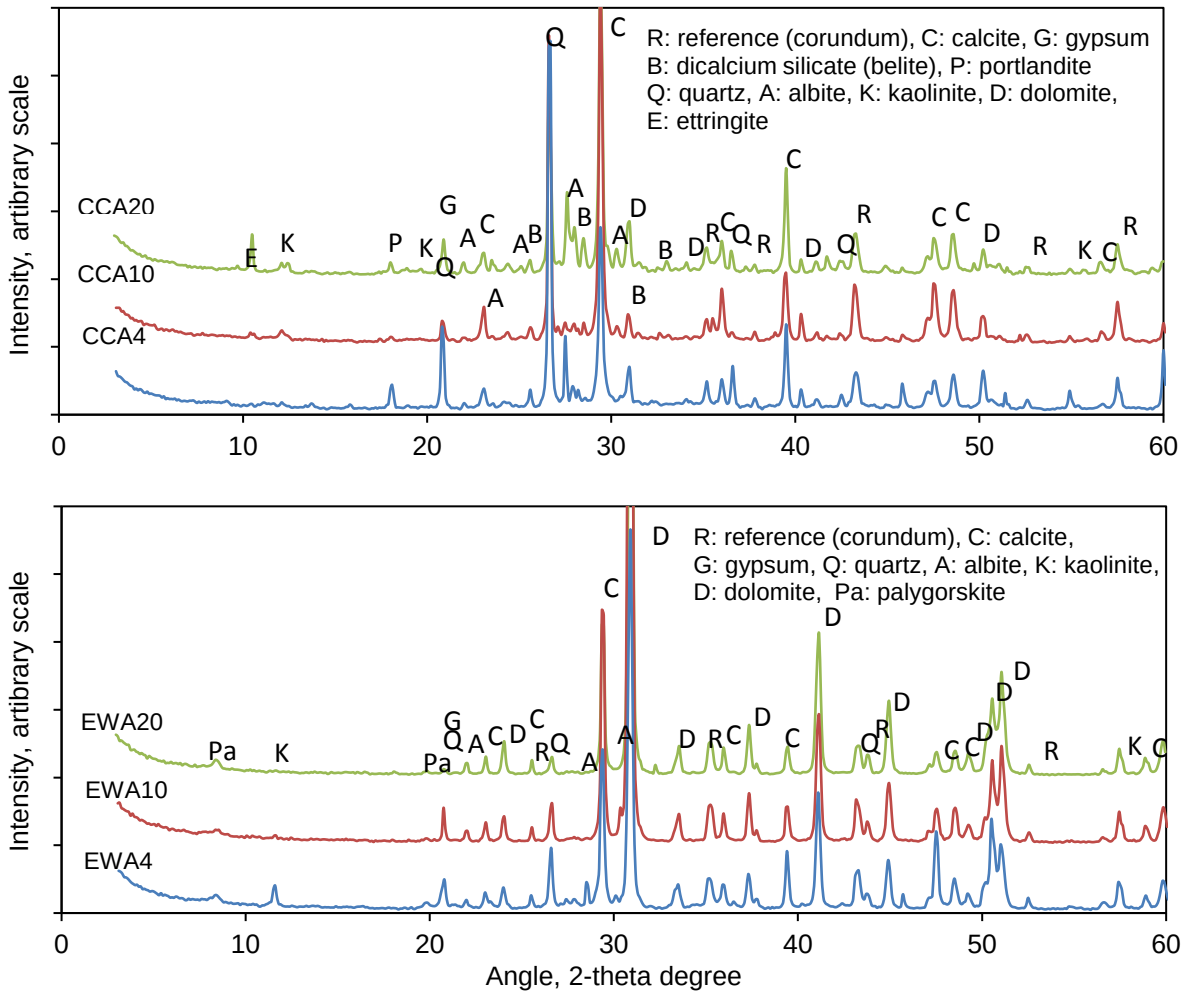


Figure 4.7: Typical XRD traces of CDWA and EWA

4.1.9 Conclusions

The results from the characterization tests, indicate that all coarse recycled aggregates obtained from Qatar (both CDWA and EWA) can be classified as CCA in accordance with BS EN 12620 (BSI, 2002) and QCS5-2 (QS, 2014) or as Type A in accordance with BS EN 206 (BSI, 2013), respectively. This classification suggests that the aggregates can be potentially used up to 50% for X0 Exposure Class concrete and up to 30% for XC1, XC2, XC3, XC4, XF1, XA1 and XD1 Exposure Classes, under BS EN 206-1:2013.

CDWA and EWA had 3 to 8% lower relative density, 4 to 7 times higher water absorption, 2 to 10 times higher fines content but relatively smoother spherical particles compared to the reference natural aggregates from UK and UAE. The mechanical properties of the recycled aggregates were also somewhat lower than the natural aggregates used. It is not possible to establish which aggregate type has broadly the most favourable properties, as these were found to vary a lot with size and between samples of the same designation.

In general terms, the majority of the properties were within the limits set by BS 8500-2 (BSI, 2015), BS EN 12620 (BSI, 2002) and QCS5-2 (QS, 2014), therefore making them acceptable for use in concrete. There were, however, cases were particular aggregate samples couldn't meet the requirements and further processing needs to be addressed. For instance, the fines content of the EWA10 samples was much higher from the acceptable limits set in both British and Qatar specifications and may pose problems with water demand when used in concrete. Similarly, the absorption values of most coarse CDWA and EWA recycled aggregate samples were even higher the 4% w/w limit set by QCS5-2 (QS, 2014) for use in non-structural concrete.

On the contrary, most coarse recycled aggregates have not met the requirements for acid-soluble and water-soluble sulfate contents. The exceptionally high sulfate contents (acid and water-soluble) of EWA10 samples would be prohibitive for use in concrete as may give rise to expansion reactions. Likewise, all coarse CDWA and EWA samples were exceeding the strict limit of 0.03% w/w for acid-soluble chlorides set in QCS5-2 (QS, 2014). Therefore, further processing is deemed essential to reduce concentration of potentially detrimental substances before the aggregates been used in concrete.

It should be noted that the testing regime of this study was to facilitate the wider use of recycled aggregates in concrete by determining appropriate use of recycled aggregates in a performance related approach. In other words, the aim was to identify at what replacement level the aggregate properties do not compromise durability of concrete. Thus, the limits set by various standards may not be applicable.

As mentioned in literature, both British and Qatar specifications do not include provisions for the use of fine CCA and RA in concrete, however this does not preclude their use where it is demonstrated that significant quantities of deleterious materials are not present and their suitability for a certain concrete application has been proved. Based on the test results, the extremely high water absorption characteristics of both fine CDWA and EWA samples, coupled with significant presence of chlorides and sulfates would be prohibitive for any use in concrete.

Coarse gabbro and limestone aggregates were found to have similar properties and both meet all the physical, mechanical and chemical limits of QCS5-2 (QS, 2014). Hence, it comes as no surprise that these aggregates have been used widely by the construction industry in Qatar.

4.2 Characteristics of CSAB cement

4.2.1 Microstructural Analysis

The degree of ye'elimite hydration, at both 7 and 28 days, was similar for each of the investigated gypsum contents, as the primary peak corresponding to ye'elimite (2θ = 23.72°) was similarly reduced (Figure 4.8). As anticipated, the presence of ettringite (2θ = 9.11°) is more pronounced at increasing gypsum contents, having a seemingly inverse relationship to the presence of stratlingite (2θ= 7.13°). The hydration product calcium aluminium oxide hydrate (CAH_{10}) was only observed for mixes having a lack of added gypsum (2θ= 6.24, 12.35°). The absence of AFm peaks, nominally monosulfate, in the XRD patterns is attributed to poorly formed crystalline structures, but AFm presence was suggested by our collaborative team at UoA in a similar study (Beltagui, et al., 2017) using a thermodynamic model and calculating the mass balance of aluminium present in the system.

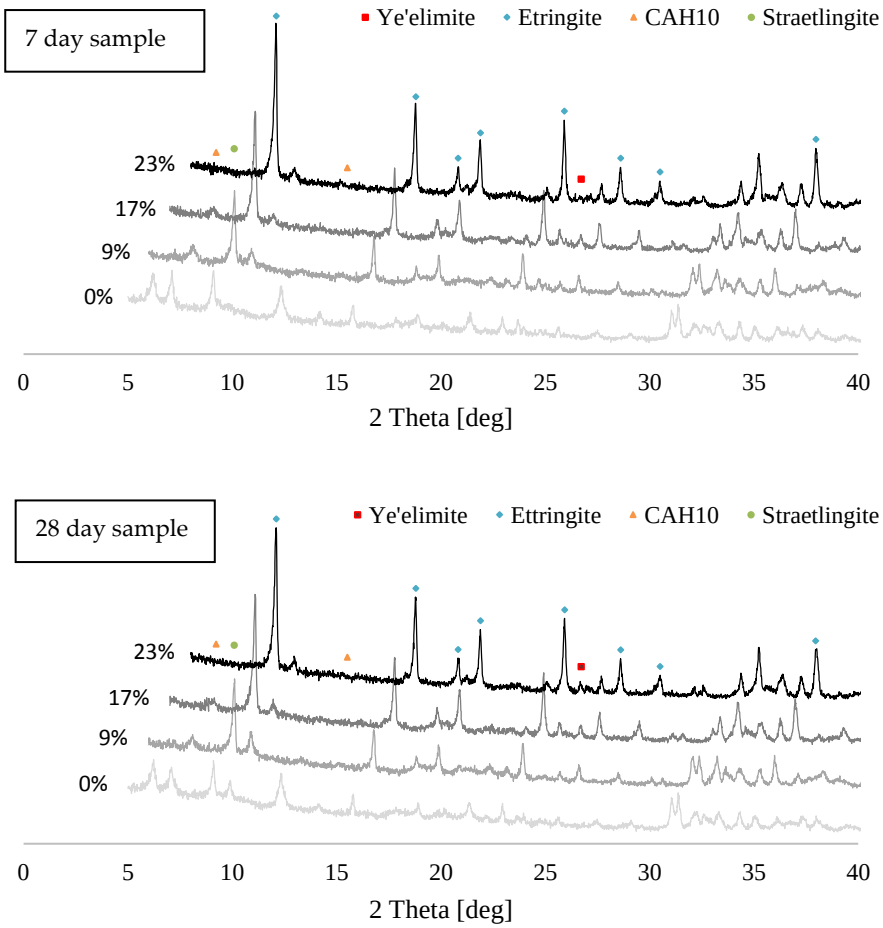


Figure 4.8: 7 and 28 day hydrated CSAB pastes of varied gypsum additions (offset for clarity)

Porosity, as determined by mercury intrusion porosimetry (MIP), has limitations as measure of in-situ performance (Winslow, 2000) but it was used here to make comparisons of the relative microstructural characteristics of the test samples (Table 4.20). Comparatively, the matrix hydrated with no gypsum addition had the lowest total porosity in a more accessible and open pore network, while conversely that with 9% produced more total porosity but with a higher degree of restricted interconnectivity. The threshold pore diameter, characterised by Winslow and Diamond (1970) as a comparative size of "choke points" limiting mercury intrusion, was found to decrease with elevated gypsum additions indicating a discontinuity in the connectivity of porosity. Since permeability of the CSAB matrix is influenced by both total porosity and the interconnectivity of said porosity, the MIP results suggest that the highest gypsum addition should produce the most impermeable matrix. This is in line with earlier findings, where researchers suggested (Telesca, et al., 2014; Garcia-Mate, et al., 2012; Bernardo, et al., 2006; Glasser and Zhang, 2001) that the rapid formation of ettringite in large amounts is capable of quickly develop a bimodal pore structure.

Table 4.20: Cumulative porosity and threshold pore diameter determined by MIP of 7 day hydrated CSAB cement pastes having varied gypsum additions

Property	Gypsum addition, % by mass of CSAB			
	0	**9**	**17**	**23**
Cumulative porosity (%)	20.3	25.1	16.3	13.3
Threshold pore diameter (nm)	186.8	48.3	18.2	14.7

4.2.2 Setting time

Setting time measurements of CSAB pastes at varying contents of gypsum are shown in Figure 4.9 for the three w/c ratios considered here. It is evident that gypsum additions, even in small increments, have a significant effect in the hydration rates of CSAB pastes. In particular, at a w/c ratio of 0.3 and no added gypsum, the initial and final set occurred at approximately 290 and 380 mins, respectively. These times were reduced by 30% at 1% addition, and dropped to 22 and 30 mins, respectively, at 2% addition. Above 2% addition, the initial and final setting times were identical regardless the amount of added gypsum.

Similar setting time patterns were also obtained at higher w/c ratios. Although the dilution effect of higher w/c ratios was apparent at no or very low gypsum additions and delayed considerably the initial and final set, this was not the case at gypsum additions above 2% where all mixtures exhibited very short setting times. Above the critical addition of 2%, the elapsed time between the initial and final set has also reduced significantly and was in the range of 8 to 15 mins. For such a short set period, the use of CSAB/gypsum combinations in concrete is almost impossible without the use of special retarding agents.

Indeed, a retarding agent compatible with CSA cements were used in the paste study to regulate the initial and final set of the CSAB cement. The agent was used at additions of 0.1 to 1.5% by weight of binder. The results showed a high inconsistence and CSAB cement appeared to be very sensitive to agent dose which in some instances resulted in final setting times greater than 24 hours. In this regard the use of the retarding agent was avoided.

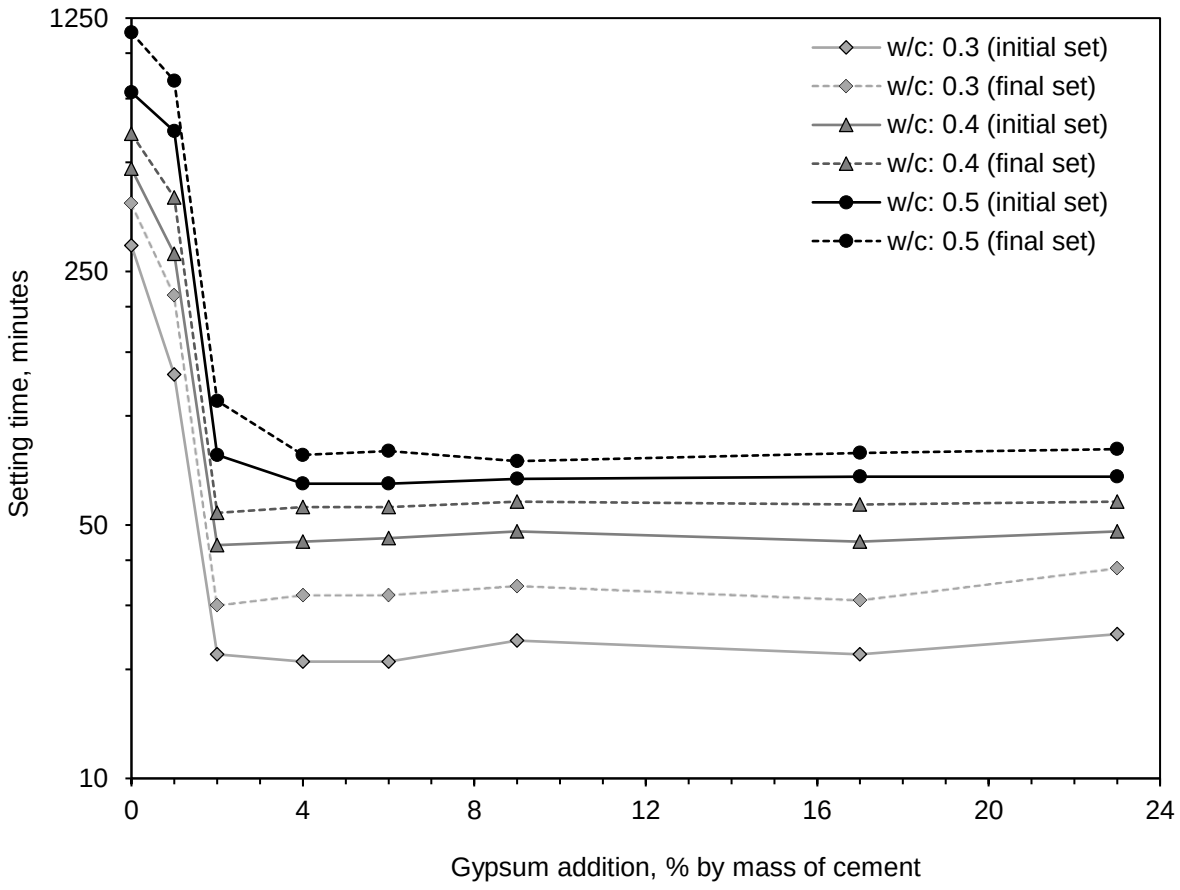


Figure 4.9: Setting times of CSAB cement pastes having varied gypsum additions in relation to various w/c ratios.

4.2.3 Alkalinity

The alkalinity of the CSAB cement having gypsum additions of 0% and 17% w/w was measured with the pore expression method and the results for the two test ages considered here are given in Table 4.21 for The pH values measured were similar to what other researchers reported (Andac and Glasser, 1999; Winnefeld and Lothenbach, 2010) and also appeared to be above the critical pH value of 11.5 for full steel reinforcement passivation. In any case however, the alkalinity of this particular CSAB cement was lower compared to a Portland Cement (typically in the range of 13-14).

Table 4.21: Alkalinity of 7 and 28 day hydrated CSAB cement pastes having varied gypsum additions

Alkalinity (pH)	Gypsum addition, % by mass of CSAB	
	0	**17**
7 day	≈11.8	≈12.1
28 day	≈12.0	≈12.2

4.2.4 Potential expansion

Dimensional changes of CSAB mortars at varying contents of gypsum are shown in Figure 4.10. With the increase of the gypsum content there is an increase in expansion. This holds true for gypsum additions up to 5%, as beyond that point the mortars exhibited similar expansion patterns with those having gypsum additions below 2%. Particularly the CSAB mortars containing gypsum at 17 and 23% w/w of cement showed after 28 days almost a dimensionally neutral pattern. The rate of expansion was also found to decrease with time. In any case however, the expansion patterns noted here are very low and are not deemed as sufficient to cause any significant cracking to the cement paste.

When the expansion patterns of the CSAB mortars are compared with that of a reference CEM I mortar containing gypsum in a such proportion that the total sulfur trioxide (SO_3) content of the mixture is 7.0 % (Figure 4.11), it can been seen that CSAB mixtures with gypsum contents below 2% and above 9% exhibited similar or lower expansion patterns. This also suggest that no significant deterioration should occur when CSAB cement is used in concrete at these gypsum addition levels.

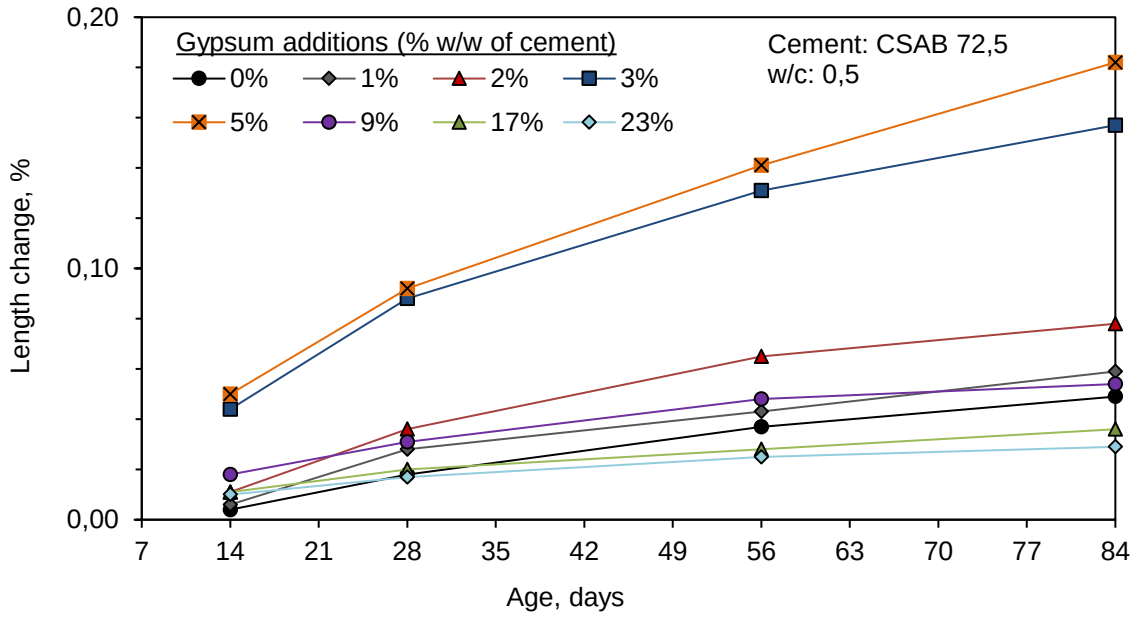


Figure 4.10: Dimensional changes of CSAB mortars at varying gypsum contents

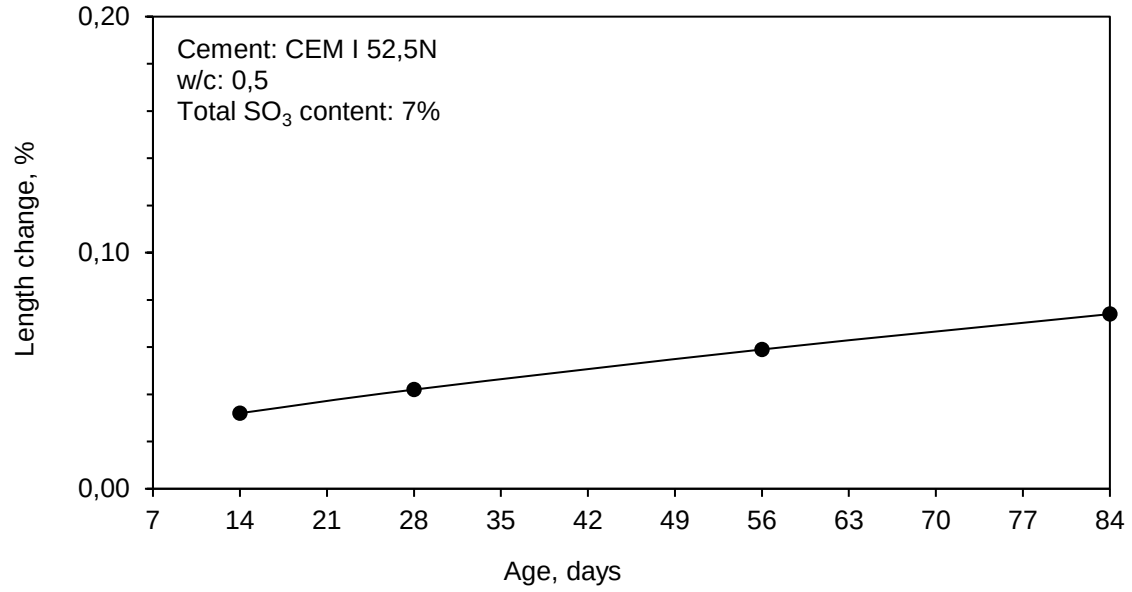


Figure 4.11: Dimensional changes of CEM I mortars having a total SO_3 content of 7%

4.2.5 Compressive strength

Compressive strength development of water-cured CSAB mortars at increasing gypsum contents is shown in Figure 4.12. As anticipated, all CSAB/gypsum combinations exhibited initially a rapid strength development and achieved approximately 95% of their ultimate strength within the first 3 days. Then, the rate of strength gaining reduced significantly and became almost neutral after 7 days.

The highest strengths were observed in CSAB mortars having gypsum additions up to 2% w/w of cement (Figure 4.13). Within that small range the addition of gypsum was found to benefit strength development. For higher addition levels, there was a massive compressive strength loss in the range of 35 to 40% regardless the amount of gypsum introduced in the mixture. This is incoherence with some previous work (Michel, et al., 2011; Hargis, et al., 2014; Trauchessec, et al., 2014;) where the strength characteristics were found to improve at elevated calcium sulfate contents. Likewise, strength follows an inverse trend in comparison with MIP observations (Table 4.20), and therefore the typical correlation of low porosity-high strength does not hold true. Since no significant expansion was noted for any mixture combination, neither any cracking or surface degradation, the strength loss is suggested to be a product of the significant difference in hydrated phase assemblages and the low density of ettringite.

Ioannou et al. (2014), who also noted a strength reduction at increased sulfate contents, have suggested that the hydration of pure CSA, in which the ye'elimite content is highest compared to other formulations, may result to a set of hydration products, most probably C-A-H, $Al(OH)_3$ and small amounts of ettringite due to absence of calcium sulfate, which could potentially have a greater effect on strength development from that of ettringite within a CSA/calcium sulfate system.

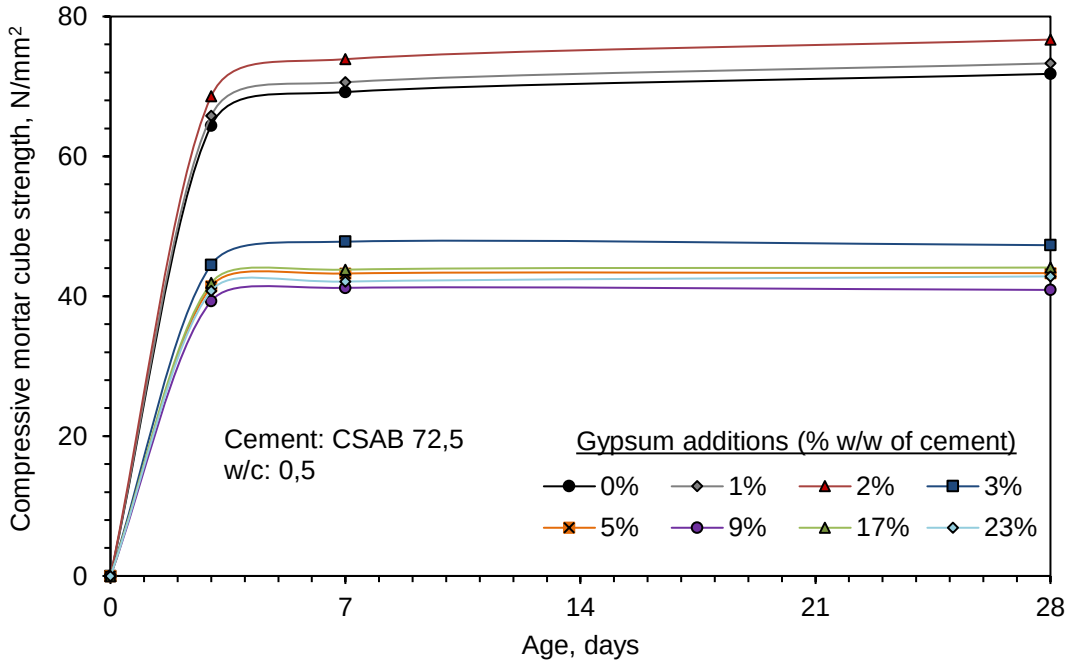


Figure 4.12: Compressive strength development of CSAB mortars at varying gypsum contents

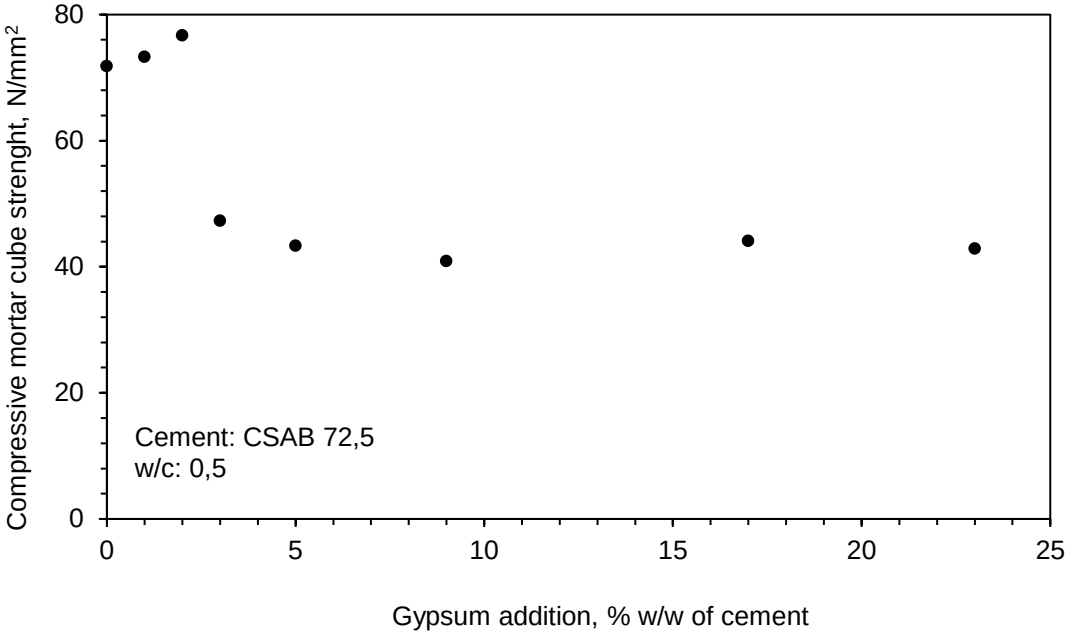


Figure 4.13: Compressive strength of CSAB mortars against gypsum content

4.2.6 Chloride ingress

Chloride ingress, based on the migration-induced accelerated protocol of NT Build 492 (Nordtest, 1999), was measured using silver nitrate staining and the corresponding calculated non-steady-state migration coefficients (D_{nssm}) are given in Table 4.22. The results indicated that the mortars fabricated with high gypsum additions, greater than 17% w/w, had the highest resistance to chloride ingress. It was noted that for the mixture containing 9% w/w gypsum, the standard test voltage (30V) produced a higher flowing current and thus for this sample, the applied voltage was lowered to 25V. This is because that mixture had the highest porosity (Table 4.20). The greater non-steady-state migration coefficient was measured in the mixture with no gypsum.

Table 4.22: Non-steady-state migration coefficients of CSAB mortars having varied gypsum additions

Test parameters	Gypsum addition, % by mass of CSAB			
	0	9	17	23
Test Voltage (V)	30	25	30	30
Duration (Hrs)	24	24	24	24
D_{nssm} ($x10^{-12}$ m^2s^{-1})	6.86	3.70	2.47	2.80

The validity of these measurements is questionable due to the short testing period of the migration method in which the potential binding affinity of the hydrates is restricted. While the amounts of crystalline AFm hydrates are not readily apparent from the XRD spectra (Figure 4.8), it can be inferred that the balance of reacted ye'elimite that does not go towards forming ettringite is likely to be contributing to the AFm content, as proposed by the thermodynamic modelling (Beltagui, et al., 2017). An approximation of the AFm content can therefore be made from the ettringite content. Using a gypsum content of 23% w/w, the ettringite intensity for any other calcium sulfate addition rate, taking into account the dilution effects of calcium sulfate, is inversely proportioned to the quantity of chloride binding hydrates.

This binding capacity of CSAB mortars having varied gypsum contents was examined using the long-term protocol of NT Build 443 (Nordtest, 1995). The results, with curve-fitted effective transport coefficients (D_e), are given in Figure 4.14 and show the relative performance in contrast to the data given in Table 4.22. Specifically addressing the profile shape of chloride content, the shallow slopes of mixtures with 9, 17 and 23% w/w gypsum contents represent significant ion mobility achieved towards equilibrating the concentration gradient of chloride ions. In contrast, the mixture with no gypsum is noted for the steepened slope noted from the surface to a depth of 30 mm depicting a restricted movement of ions once entering the cement matrix. As porosity measurements (Table 4.20) do not exhibit a correlated difference, the reason for this is probably due to chloride binding.

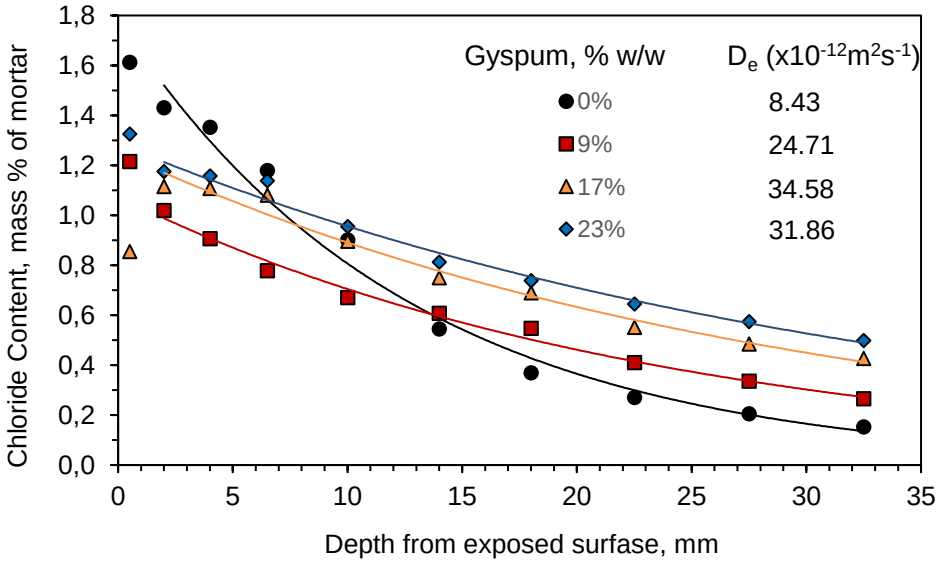


Figure 4.14: Chloride profiles of CSAB mortars with varied gypsum additions

The results also highlight an important distinction between the different test methodologies used for determining apparent diffusion coefficients. The data arising from the short duration, migration-accelerated methodology of NT Build 492 (Nordtest, 1999), shows that for high sulfate mixtures, pore refinement is the property controlling ion mobility. These results and the associated interpretation is in agreement with Ioannou et al. (2015), who resulted in improved chloride diffusion resistance owing to microstructural densification via preferred ettringite formation in a ternary calcium sulfoaluminate-anhydrite-fly ash cement. However, the long duration result, using the concentration-accelerated methodology of NT Build 443 (Nordtest, 1995), suggests that the restriction of chloride ion ingress is possibly via hydrate conversion and binding. This has significant impact on the selection of standard test methods to characterise the real world durability performance of CSA concrete.

4.2.7 Conclusions

The research data presented in this study was aimed at determining an optimum calcium sulfate content available during hydration for the purpose of enhancing the material's properties. It is recognised that the test results presented here are representative of testing conducted on a limited number of samples of each mix type tested for each condition assessed, however the performance discussed herein is sufficient to indicate the required gypsum addition when using this particular CSAB in concrete.

The properties of the CSAB cement were shown to be much influenced by the gypsum content. As expected, ettringite formation was noted to increase with gypsum addition. Similarly, an initial addition of gypsum led to a slight increase in the total porosity which was then reduced noticeably with any further additions. Conversely, setting time, potential expansion, strength development and chloride ingress displayed an inverse non-linear dependence on gypsum.

Setting times of gypsum-free systems were found sufficient and comparable to Portland cements. When gypsum was introduced in the mix, setting times were reduced significantly and for additions greater than 2% by mass of cement the resulting setting times were similar regardless the amount of added gypsum. In the latter case neither the increase of free water in the system appeared to delay much set of cement. Likewise, gypsum additions up to 2% helped the CSAB system to achieve its potential maximum strength, with higher contents resulting in strength losses around 40%. No correlation was found between the measured total porosity of samples and the strengths observed, with higher porosity corresponding to lower strengths.

From a durability point of view, the expansion rates of mortars at all investigated levels of calcium sulfate were found within a reasonable range not adequate to cause any cracking, with the mixes having gypsum additions below 2% or above 9% to experience similar expansion patterns with the reference PC sample. The alkalinity of the CSAB cement was lower than PC but above the critical pH value of 11.5 for full steel reinforcement passivation. With regards to chloride ingress in a belite calcium sulfoaluminate cement, the resistance should be determined using test methods of sufficiently long duration to allow binding to form, as rapid test methods may give misleading results. In the former case, binding capacity appeared to be inversely proportional to the total calcium sulfate content.

Overall, the calcium sulfate content does influence the properties of the CSAB cement and it is likely that the optimum for full ye'elimite hydration or porosity would not be coincident for strength or chloride ingress. The present data suggests a gypsum addition of no more than 2% by mass of the CSAB cement to achieve maximum strength, enhance chloride resistance, reduce expansion potential and prevent rapid setting.

CHAPTER 5: **RECYCLED AGGREGATE CONCRETE**

This Chapter investigates the effect of Qatari recycled coarse aggregates on fresh, engineering and durability properties of concrete. The concretes considered are those described in Section 3.5.1 and tested as outlined in Section 3.5.4. The concrete properties were compared at both equal w/c ratio and strength basis and the results were related to literature findings. Correlations between different properties were also considered to explain particular trends. The properties of recycled aggregate concretes were then compared with those of control concrete mixes and the validity of the current study was established.

5.1 **Fresh properties of concrete**

The fresh properties of all CEM I-based recycled aggregate concretes are shown in Table 5.1 to Table 5.3. The plastic density measurements gave general agreement with the sum of the constituents (i.e. theoretical density ± 20 kg/m^3) for all concrete mixes and this provides confidence that the concretes were batched correctly. Given that the concrete mixes were designed for a fixed water content of 185 lt/m^3 and an equal consistence i.e. BS EN 206-1 (BSI, 2013) consistence class S3 with a nominal slump between 100-150mm, the superplasticizer (SP) dosage was adjusted to achieve the required consistence. The SP dose was found to vary in the range of 0.27- 0.7% by mass of cement (recommended limits by the supplier: 0.25 - 1.0%), with the 0.75 w/c ratio dose at the higher end of this. The latter effect was due to higher fine aggregate contents resulting in a greater surface area.

For a single w/c ratio and the same type of recycled aggregates, the SP was kept constant to examine the effect of recycled aggregates to the consistence of concrete. For the main concrete series, where coarse granite aggregates used as a reference material, the incorporation of elevated amounts of CDWA was generally found to improve the consistence of the concrete. Given that the water absorption of recycled aggregates was treated properly, the beneficial effect of CDWA observed in relation to measured slump values can be attributed to the more spherical shape of particles compared to granite aggregates. The flakiness index of granite aggregates was higher than that of CDWA (Table 4.14 and Table 4.15) and as noted in other studies (Bouquety, et al., 2007; Chidiroglou, et al., 2008; Fonseca, et al., 2011), when the portion of flat particles in concrete increases, the workability decreases.

A similar relationship between aggregate FI values and concrete consistence can be obtained if the consistence of the reference CEM I GRA concrete is compared to that of control CEM I GA and CEM I LA concretes. The control concretes required less SP dose for the same consistence class due to the more spherical particle shape of limestone and gabbro aggregates. When coarse CDWA replaced gabbro or limestone aggregates in the control concrete series similar values were measured for the same SP dose.

On the contrary, to maintain the consistence of the mixes containing EWA within the same range, the SP dose (compared to reference concretes) had to increase by at least 0.1% per mass of cement for every 15% v/v increase in the content of EWA. This was probably due to the high fines content of EWA owing to increase the surface area and aggregate water absorption which may has been underestimated as it was determined on a washed sample with no silt present. As shown in Table 4.4 and the amount of fines present in 4/10mm and 10/20mm EWA were on an average level of approx. 4.6% and 7.3% by weight, respectively; percentages considerably higher from those of NA or CDWA or that allowed by both the British and Qatari Standards. It is evident therefore that with no proper screening or washing to remove the excess of fines, the use of EWA in concrete, even in small replacement levels, will be problematic. A reduction of fines content was initially considered but then it was considered that screening was not practically feasible and washing would change the water soluble chloride and sulfate content of the material.

In line with expected behaviour, the use of pozzolanic additions as binary cement combinations in the mix design improved slightly the consistence of concretes (Table 5.3).This beneficial effect is generally attributed to the spherical shape of fly ash particles or to the smooth glassy texture of GGBS particles, both having a lubrication effect in the mix (Thomas, 2013). The incorporation of small amounts of CDWA, as noted previously, had no effect to the consistence of these concrete.

Bleeding, segregation, compatibility, finishability and cohesiveness were all assessed qualitatively based on visual observations during placement and compaction of concrete and found generally unaffected, with the exception of mixes containing 30% EWA. The insufficient consistence of these mixes resulted in extended compaction periods and also air to be trapped between the mould face and the concrete surface forming numerous blowholes at the outer concrete surfaces.

Table 5.1: Fresh properties of CEM I recycled aggregate concretes

Mix Name	w/c ratio	Plastic density, kg/m³	SP[1], % of cement content	Slump, mm	Bleeding	Segregation	Finishability	Compactability	Cohesiveness
CEM I GRA	0.45	2385	0.41	105	None	None	Good	Good	Good
CEM I GRA30CDWA		2370	0.41	100			Good	Good	
CEM I GRA50CDWA		2360	0.41	130			Good	Good	
CEM I GRA15EWA		2385	0.50	110			Good	Good	
CEM I GRA30EWA		2390	0.50[2]	60			Low	Low	
CEM I GRA	0.55	2385	0.47	100	None	None	Good	Good	Good
CEM I GRA30CDWA		2370	0.47	110					
CEM I GRA50CDWA		2345	0.47	145					
CEM I GRA15EWA		2370	0.58	110					
CEM I GRA30EWA		2365	0.58[2]	95					
CEM I GRA	0.65	2380	0.45	105	None	None	Good	Good	Good
CEM I GRA30CDWA		2360	0.45	120	Slight		Good	Good	Good
CEM I GRA50CDWA		2355	0.45	140	Slight		Good	Good	Average
CEM I GRA100CDWA		2325	0.45	125	None		Good	Good	Good
CEM I GRA15EWA		2375	0.70	100	None		Good	Average	Good
CEM I GRA30EWA		2365	0.70[2]	70	None		Average	Low	Good
CEM I GRA	0.75	2380	0.53	110	Slight	None	Good	Good	Good
CEM I GRA30CDWA		2350	0.53	105			Good	Good	Good
CEM I GRA50CDWA		2345	0.53	100			Good	Good	Good
CEM I GRA100CDWA		2310	0.53	120			Good	Good	Average
CEM I GRA15EWA		2365	0.70	100			Average	Average	Good
CEM I GRA30EWA		2365	0.70[2]	65			Low	Low	Good

[1] The superplastiser dose was adjusted to achieve S3 slump class

[2] The superplastiser dose was kept constant for mixes containing EWA to highlight the effect of this aggregate type on concrete consistence

Table 5.2: Fresh properties of Control recycled aggregate concretes

Mix Name	w/c ratio	Plastic density, kg/m³	SP dose[1], % of cement content	Slump, mm	Bleeding	Segregation	Finishability	Compactability	Cohesiveness
CEM I LA	0.55	2400	0.32	160	None	None	Good	Good	Good
CEM I LA50CDWA		2375	0.32	155	None	None	Good	Good	Good
CEM I LA30EWA		2390	0.42	95	None	None	Good	Good	Good
CEM I GA	0.55	2480	0.27	160	None	None	Good	Good	Good
CEM I GA50CDWA		2405	0.27	165	None	None	Good	Good	Good
CEM I GA30EWA		2425	0.35	100	None	None	Good	Good	Good

[1] The superplastiser dose was adjusted to achieve S3 slump class

Table 5.3: Fresh properties of CEM I/FA and CEM I/GGBS recycled aggregate concretes

Mix Name	w/c ratio	Plastic density, kg/m³	SP dose[1], % of cement content	Slump, mm	Bleeding	Segregation	Finishability	Compactability	Cohesiveness
CIVB-V GRA	0.45	2360	0.3	110	None	None	Good	Good	Good
CIVB-V GRA30CDWA		2340	0.3	115	None	None	Good	Good	Good
CIVB-V GRA	0.55	2355	0.3	105	None	None	Good	Good	Good
CIVB-V GRA30CDWA		2335	0.3	100	None	None	Good	Good	Good
CIIIA GRA	0.45	2390	0.4	105	None	None	Good	Good	Good
CIIIA GRA30CDWA		2375	0.4	110	None	None	Good	Good	Good
CIIIA GRA	0.55	2375	0.4	110	None	None	Good	Good	Good
CIIIA GRA30CDWA		2365	0.4	115	None	None	Good	Good	Good

[1] The superplastiser dose was adjusted to achieve S3 slump class

5.2 **Strength characteristics**

Compressive strength tests were carried out on 100 mm concrete cubes at test ages up to 180 days and the results for the CEM I recycled aggregate concretes are given in Figure 5.1. As expected, all concretes gave increased cube strength with reducing w/c ratio (0.75 to 0.45) and with increasing test age up to 180 days.

In general, there was a gradual reduction in the compressive strength as the recycled aggregate content increased, which was more pronounced at lower w/c ratios. This is because at low strength concretes the compressive strength depends intrinsically on the cement paste strength and to a less extend on the quality of recycled aggregates (Ryu, 2002; Poon, et al., 2004a). However, for low substitution levels up to 30% v/v, the effect of recycled aggregate content on 28-day compressive strength was minimal and within experimental variability as Figure 5.2 shows. This confirms earlier studies (Topçu and Sengel, 2004; Etxeberria, et al., 2007; Kou and Poon, 2008; Yang, et al., 2011) in which it has been suggested that up to a practical limit CCA may be used without adversely affecting concrete strength, and also seems to verify the current rule for use of CCA in designated concretes RC25 to RC50 in BS 8500-2 (BSI; 2015) which allows a maximum mass fraction direct replacement of 20% of natural coarse aggregate. For higher substitution levels of 50% and 100% v/v the results indicated a 28-day compressive strength loss of up to 12% and 17%, respectively. No difference in the fractured patterns/surfaces was noted between concretes containing or not various amounts of recycled aggregates though.

Figure 5.2 also shows the relative effect of aggregate type on 28-day compressive strength. There was little difference in the behaviour between concretes containing 30% v/v CDWA or EWA at an equal w/c ratio. The slight improved strength of the concretes containing 15% v/v EWA is probably due to the increased amount of fines resulting in a reduced free water in the mix.

The 28-day flexural strength results for the CEM I recycled aggregate concretes, measured in 100x100x50mm prismatic specimens, are shown in Figure 5.3. The flexure strength of concrete is significantly low and indeed the flexure strength accounted for 10 to 13% of the equivalent 28-day compressive strength. This results to an almost linear relationship between 28-day compressive and flexure strength with a correlation factor of 0.94. Similar observations were noted elsewhere (Koo and Poun, 2008; Xiao, et al., 2012). Similar to compressive strength data, the inclusion of coarse CDWA or EWA does have an increasing detrimental effect on flexural

strength. The strength losses noticed here were up to 13% and 20% for substitution levels up to 50% and 100%, respectively. The use of EWA reduced the flexural strength to a greater extent than that of CDWA under similar conditions. A possible explanation could be the higher fine content forming weak zones in the interface between adhesion mortar and aggregate surface.

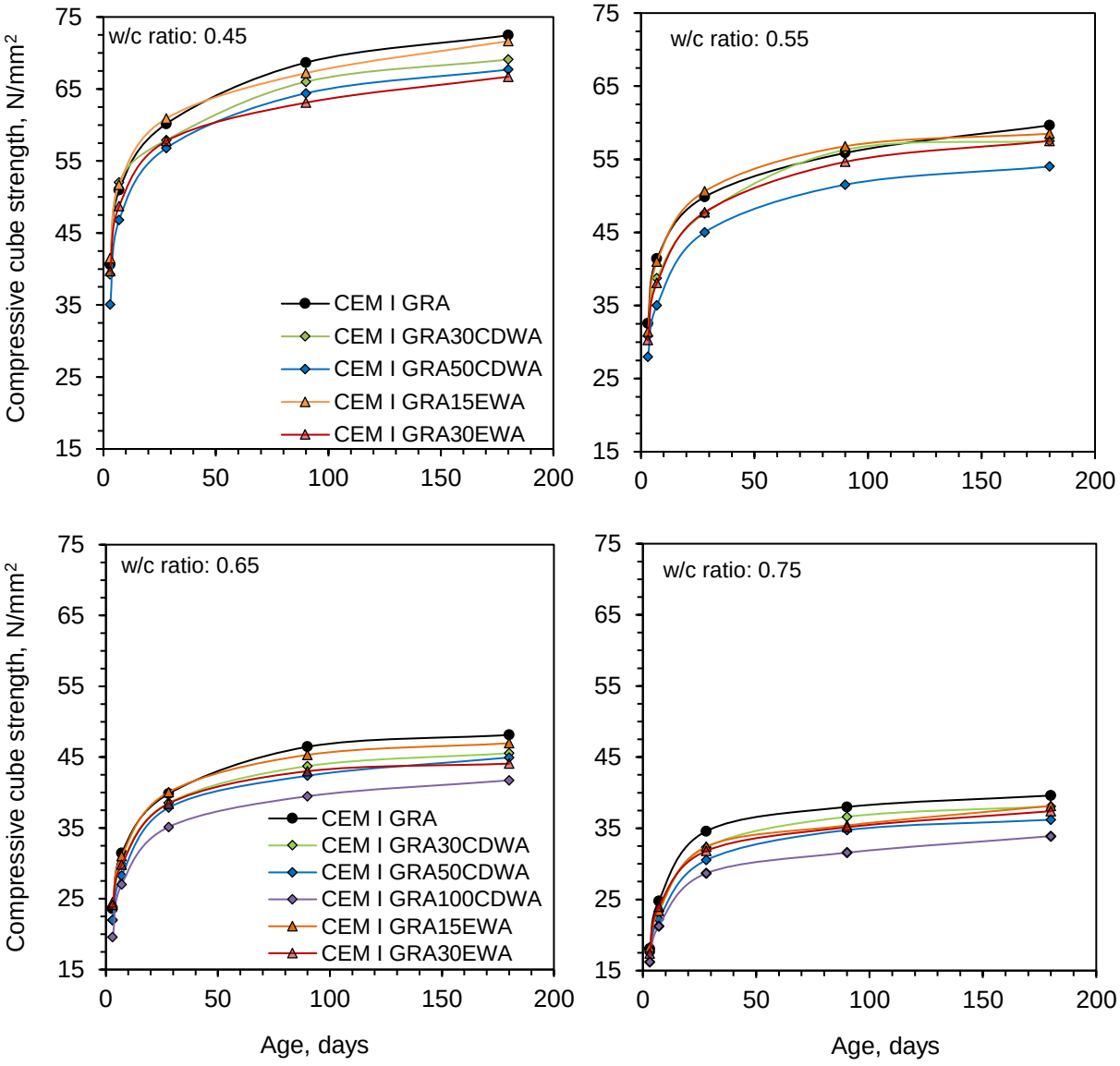


Figure 5.1: Development of compressive cube strength of CEM I recycled aggregate concretes at w/c ratios of 0.45-0.75

Figure 5.4 and Figure 5.5 show the relationship w/c ratio with 28-day compressive and flexural strength, respectively. The increasing detrimental effect of CDWA or EWA contents to concrete strength with increasing the w/c ratio is more obvious in these graphs. Since recycled aggregate concretes resulted in lower 28-day compressive strengths compared to natural aggregate concretes, particularly at replacement levels above 30% v/v, it would be necessary in practice to

reduce the w/c ratio of the mix to achieve equal strength. This has been well demonstrated in earlier studies (Dhir, et al., 1999; Pain and Dhir, 2010), however, it should be noted that from a practical point of view, this may result in unsustainable mixes beyond a certain level of reduction in w/c, owing to the requirement for higher cement contents and/or the need for increased SP dosages to offset lower water contents.

When the 28-day strength characteristics of CEM I recycled aggregate concretes were compared with those of control recycled aggregate concretes at a w/c ratio of 0.55, the results showed an identical performance (Figure 5.6 and Figure 5.7). This provides confidence that the rest of the results will be similar to that of control concrete series.

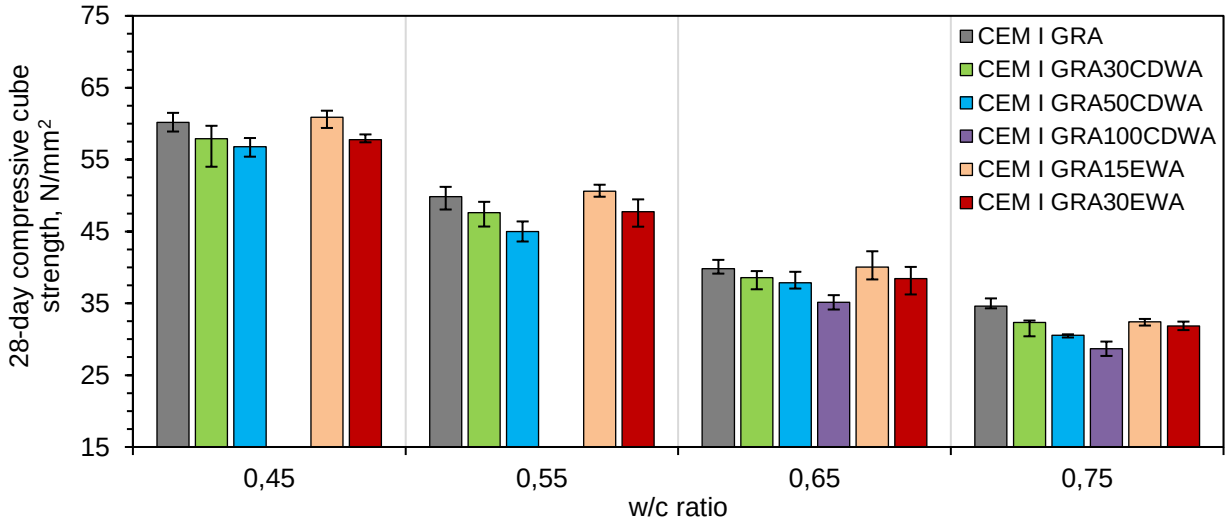


Figure 5.2: Comparison of 28-day compressive cube mean strength of CEM I recycled aggregate concretes at w/c ratios of 0.45-0.75 (error bars show min and max values)

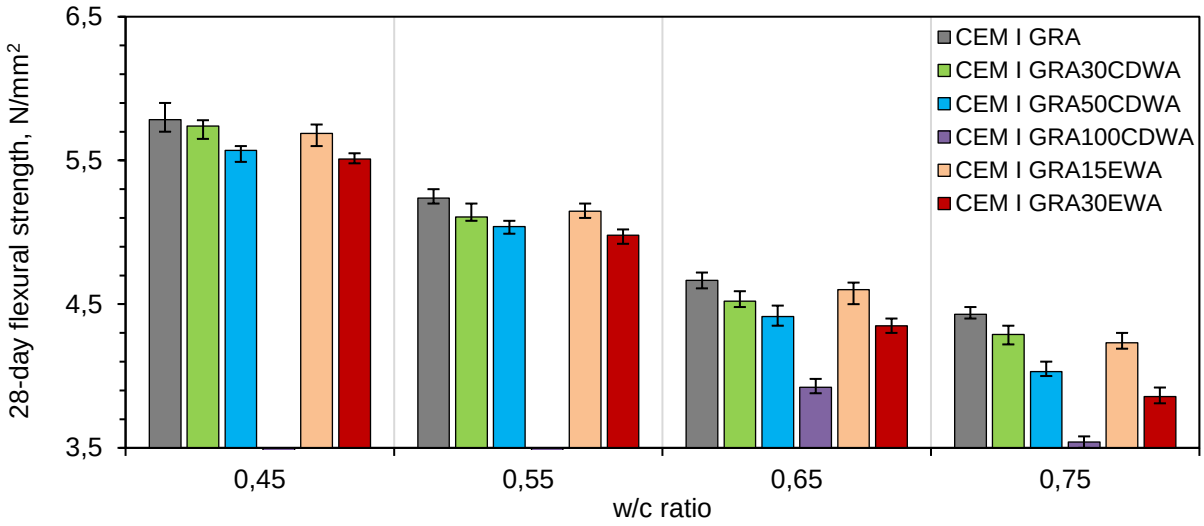


Figure 5.3: Comparison of 28-day flexural mean strength of recycled aggregate CEM I concretes at w/c ratios of 0.45-0.75 (error bars show minimum and maxium values)

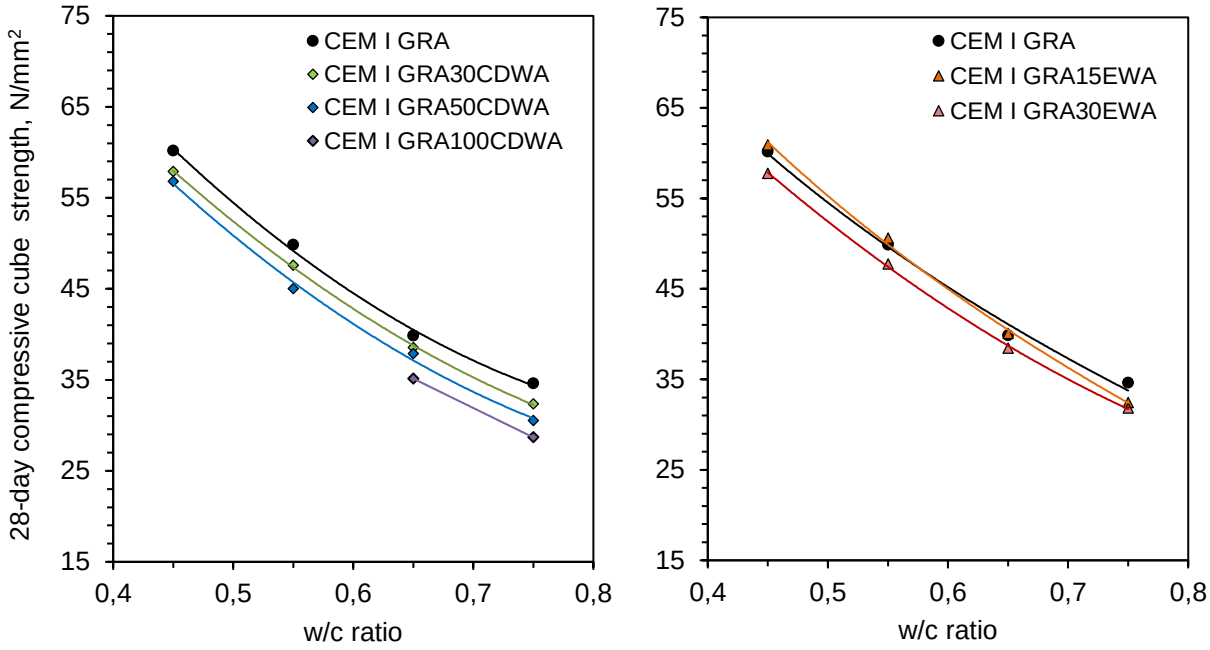


Figure 5.4: Relationship between 28-day compressive cube strength and w/c ratio for the test CEM I recycled aggregate concretes

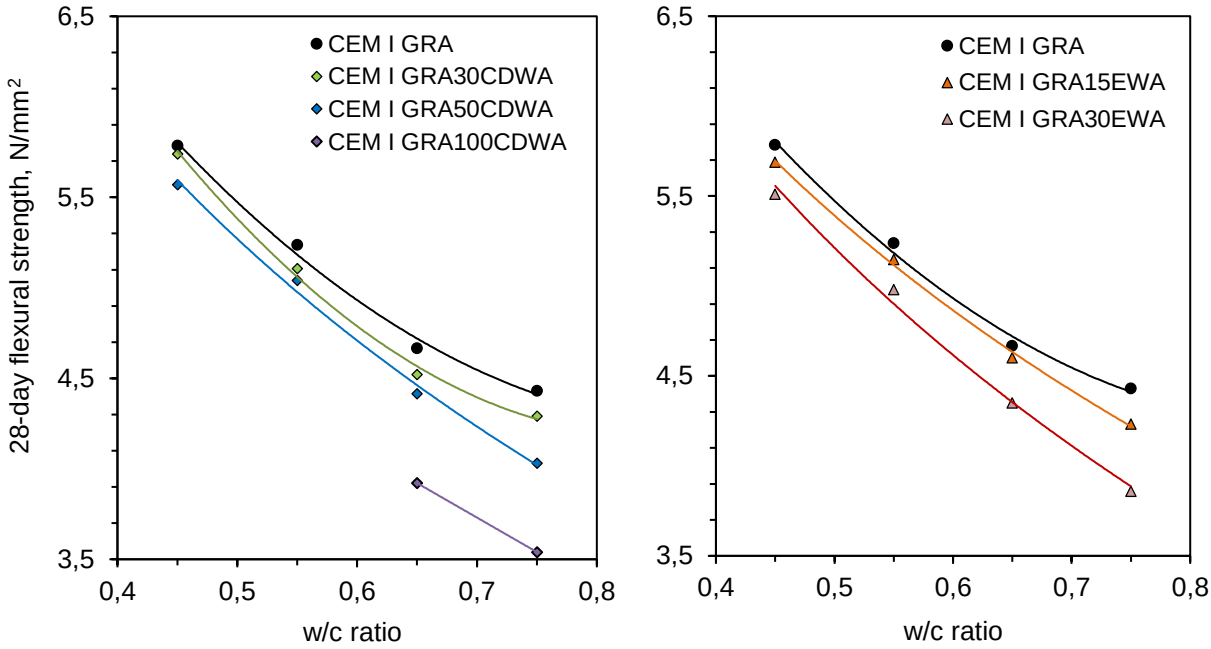


Figure 5.5: Relationship of 28-day flexural strength and w/c ratio for the test CEM I recycled aggregate concretes

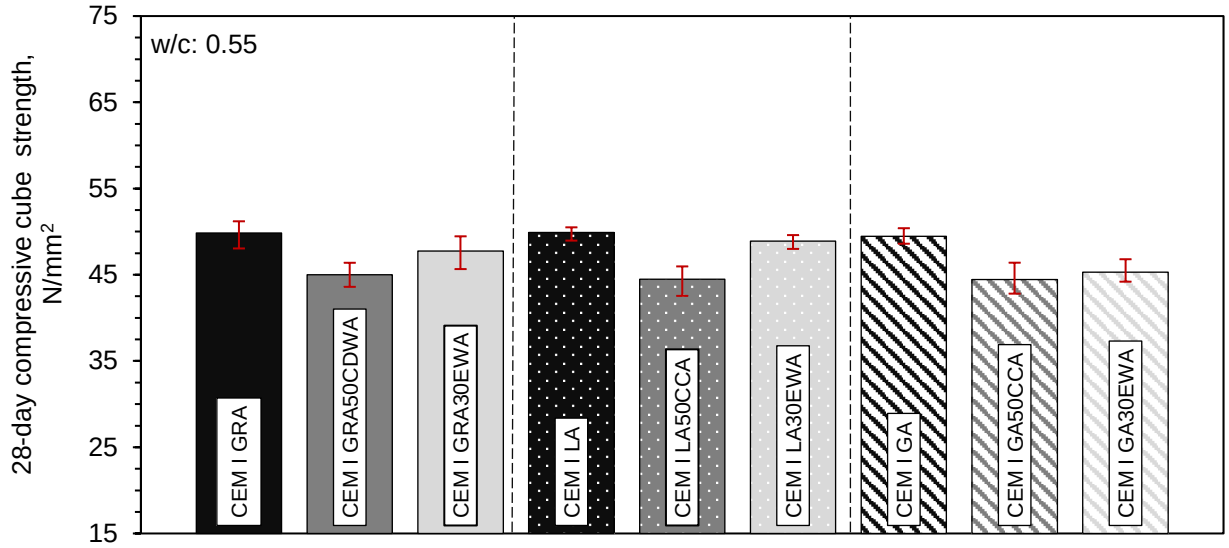


Figure 5.6: Comparison of 28-day compressive cube mean strength of CEM I and control recycled aggregate concretes at a w/c ratio of 0.55 (error bars show min and max values)

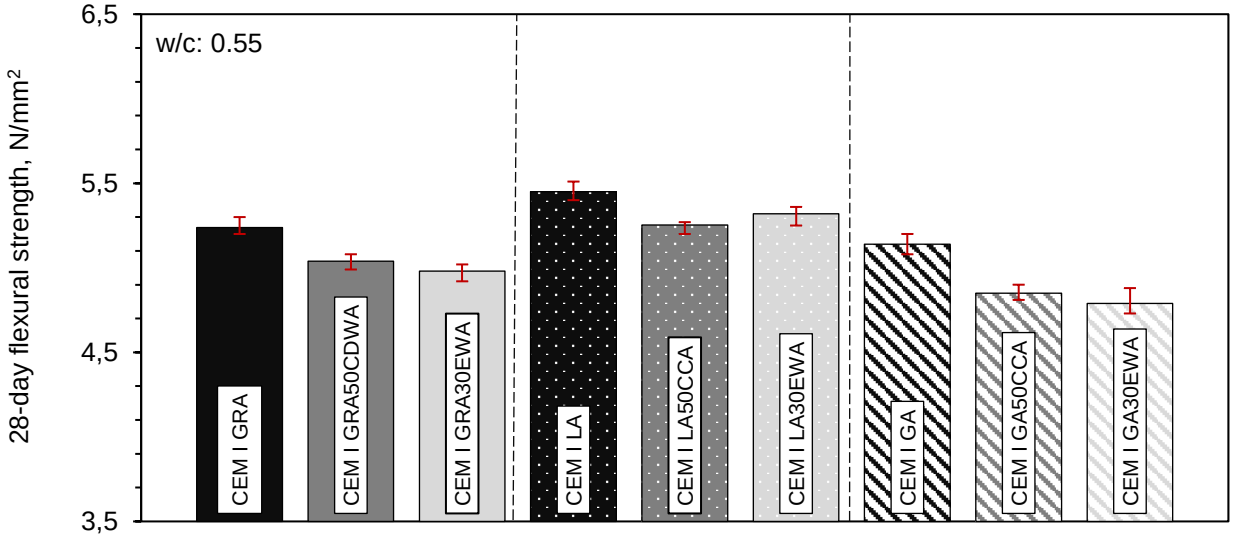


Figure 5.7: Comparison of 28-day flexure mean strength of CEM I and control recycled aggregate concretes at a w/c ratio of 0.55 (error bars show min and max values)

The results for cube strength development of recycled aggregate concretes produced with binary cement combinations are shown in Figure 5.8. Similar to previous results, only a minor reduction to cube strength at all test ages was noted for mixes incorporating recycled aggregate up to a replacement level of 30% v/v. Figure 5.9 gives the relationship of compressive and flexural strength of these concretes with w/c ratio. The CEM I GRA and CEM I GRA30CDWA were also included in the graphs for comparison purposes. In line with expected behavior, for the same w/c ratio and content of cementitious material (CEM I + Pozzolana) the resulting strengths were lower. To achieve an equivalent strength with that of CEM I concretes at 28 days

it is necessary to adjust the w/c ratio by increasing the cementitious content, as the free water content remains constant for all mixes. Although GGBS was used in higher substitution level (50% instead of 40% w/w for FA), Figure 5.9 suggests a w/c reduction of only 0.05 compared to 0.2 in case of CIIIA concretes for an equivalent strength performance. This is due to the higher pozzolanicity of GGBS which have been well documented in the literature (The Concrete Society, 2011; Neville, 2012).

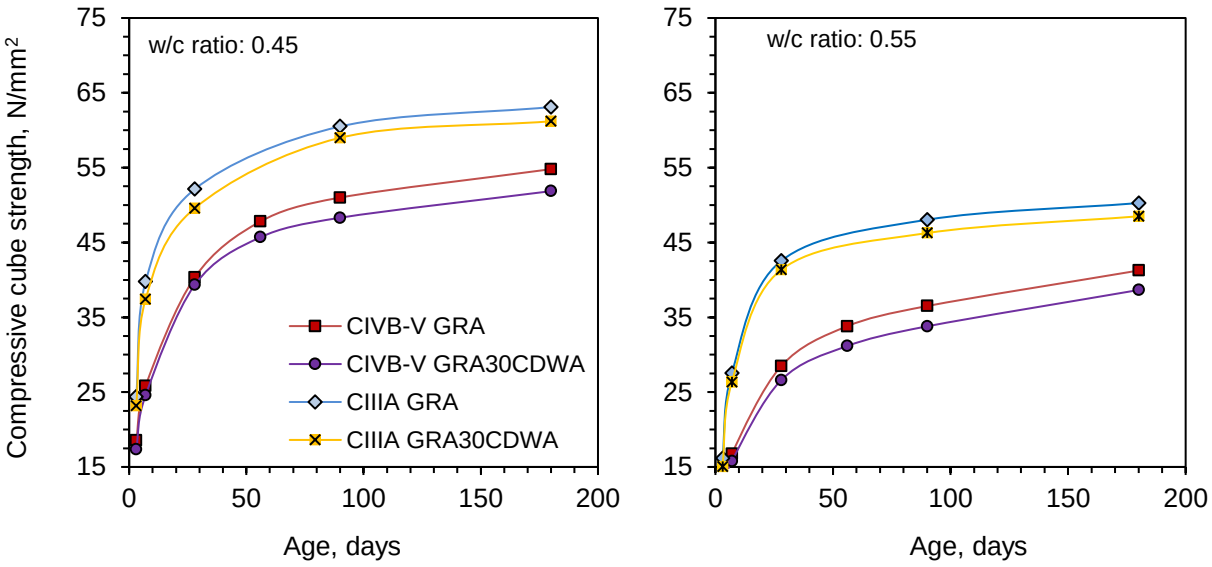


Figure 5.8: Development of compressive cube strength of CEM I/FA and CEM I/GGBS recycled aggregate concretes at w/c ratios of 0.45 and 0.55

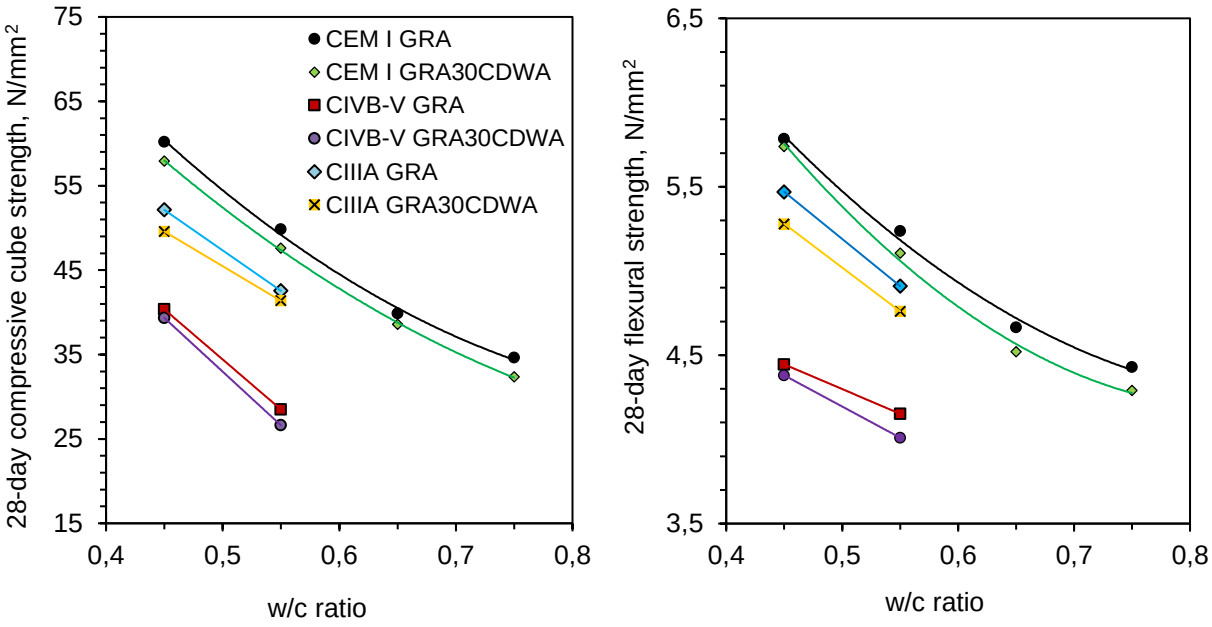


Figure 5.9: Relationship between 28-day strength and w/c ratio for the test CEM I/FA and CEM I/GGBS recycled aggregate concretes

5.3 **Drying shrinkage**

Drying shrinkage tests were carried out for selected mixes and the results are presented in Figure 5.10. The drying shrinkage of test specimens was found to increase with time and stabilise at about 90 days. At a given time, drying shrinkage appeared to increase with w/c ratio and reduce with concrete strength. This is better demonstrated in Figure 5.11 which shows the drying shrinkage in relation to w/c ratio and concrete strength. According to literature (Hansen and Boegh, 1985; Limbachiya, et al., 2000; Sagoe-Crentsil, 2001), concrete shrinkage is primarily due to shrinkage of cement paste and is affected by the proportions of the components, amount of free water, moisture while curing, dry environment and member size. Given that the sample size, curing conditions and drying environment were identical, the higher shrinkage at higher w/c ratios could be explained by the material proportions and the content of free water. At higher w/c ratios, the paste volume is less but there is more free water to evaporate (Figure 5.10) and the paste matrix is more susceptible to volume changes due to reduced stiffness.

Unlike the paste effect described above, the presence of aggregates in the system tend to reduce the concrete shrinkage because of restraining the paste shrinkage. The mechanical properties of construction aggregates, such as elastic modulus and shrinkage itself can influence concrete shrinkage but in most cases this is deemed to be a secondary effect. (Goto and Fujiwara 1976). This supports current findings where recycled and natural aggregate concretes displayed similar trends with regard to the rate and amount of shrinkage (Figure 5.10), given that both CDWA and EWA have met the 0.075% aggregate drying shrinkage limit set in BS EN 12620 (BSI, 2002).

Figure 5.11 also shows that the incorporation of 50% v/v CDWA had a slight detrimental effect to drying shrinkage compared to reference concrete, possibly due to their lower restraining capacity compared to granite aggregate particles. This effect appears to be greater from that of w/c ratio. On the contrary, similar or marginally lower expansion values were obtained for concretes containing EWA at a 30% v/v replacement level. This behaviour can be attributed to the higher fines content resulting in a reduced w/c ratio. In both cases however, the shrinkage profiles at early ages are nearly identical to those observed for reference concrete and any effect was mainly noticeable at later ages and at higher w/c ratios. As expected, the water evaporation recorded for mixes containing recycled aggregates was greater (Figure 5.10), owing to the higher water absorption values of CDWA and EWA compared to granite aggregates.

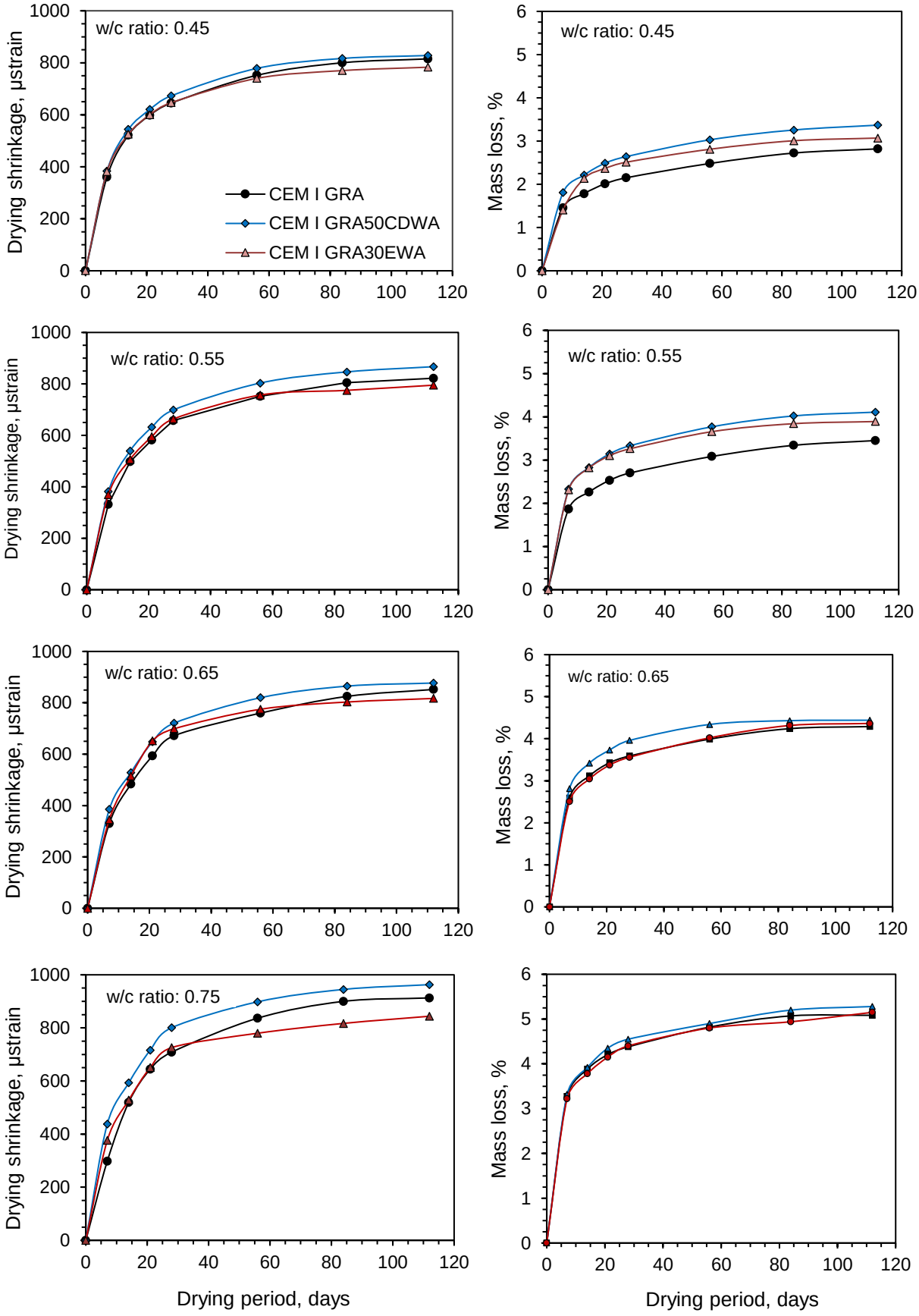


Figure 5.10: Concrete shrinkage development of selective CEM I recycled aggregate concretes alongside with water evaporation at w/c ratios of 0.45-0. 75.

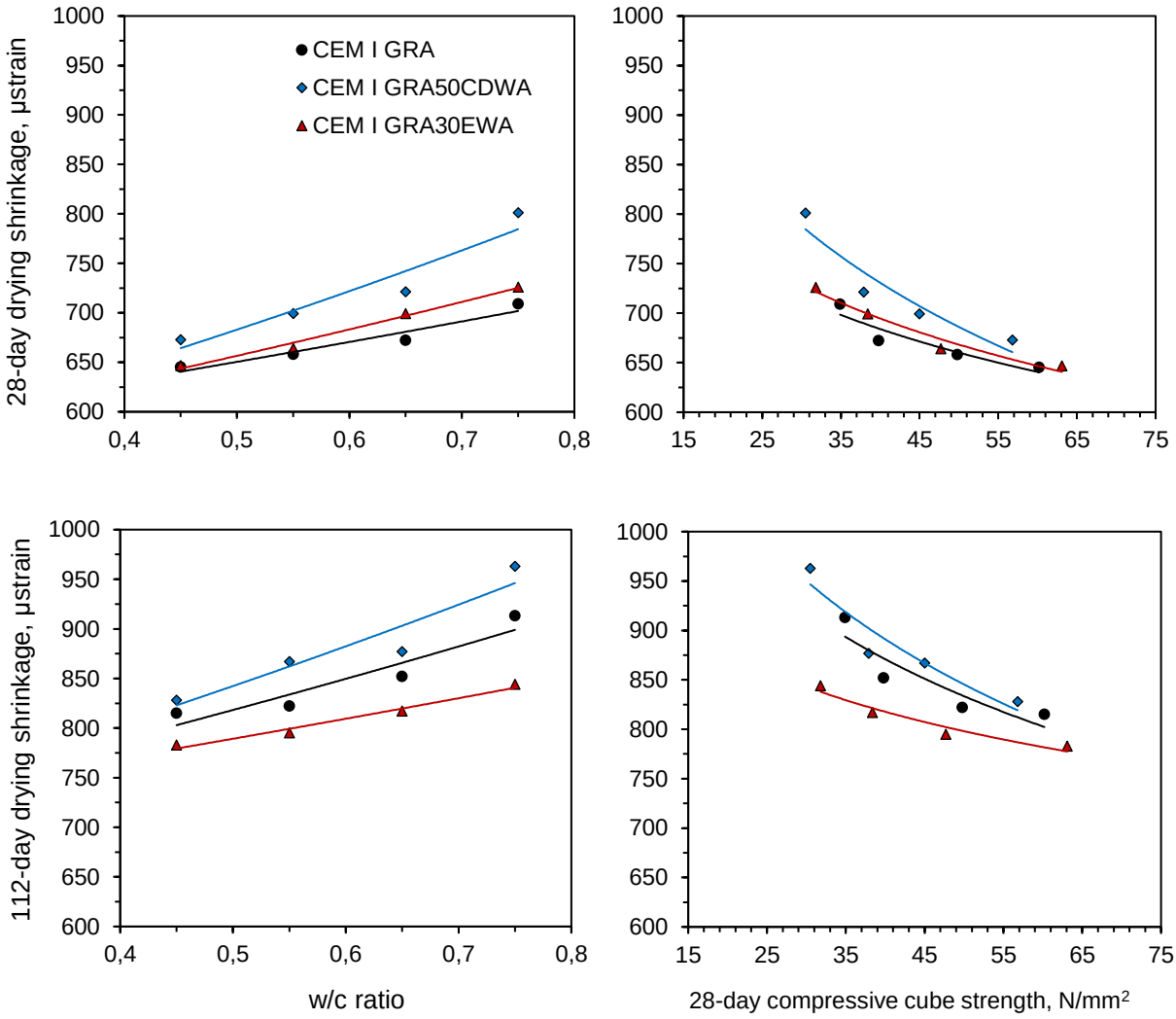


Figure 5.11: Drying shrinkage at 28 and 112 days of selective CEM I recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength.

5.4 **Absorption**

The process of water absorption in recycled aggregate and natural aggregate concretes are similar and in general obey the same laws (Levy and Helene 2004). Initial surface absorption test results for recycled aggregate concretes after 10 mins (ISAT-10) are shown in Figure 5.12. It can be seen that up to 30% v/v aggregate replacement there was no significant effect of recycled aggregate content, and thereafter ISAT-10 increased with recycled content. This comes in agreement with earlier findings (Dhir, et al., 1998; Kwan, et al., 2012). Mixes produced with CDWA contents of 50% and 100% v/v displayed ISAT-10 values increased by at least 22% and 31%, respectively. This increase is thought to be due to the increased proportion of cement paste attached to CDWA and serves as a potential conduit for moisture transport. On the contrary, the inclusion of EWA to a 30% v/v replacement level generally resulted in lower ISAT-10 values.

As mentioned previously, the underestimated water absorption value of EWA due to increased portion of fines is believed to have potentially reduced the w/c ratio and the resulting porosity. The ISAT-10 values were also found identical to those obtained for control recycled aggregate concretes (Figure 5.13).

The Concrete Society in Technical report No. 31 gives guidelines on the water absorption performance of concretes (The Concrete Society, 2008). Comparison of the test data with the limits suggested in Table 3.2 of the report revealed that all concretes containing recycled aggregates less than 30% v/v had an average water absorption throughout the range of w/c ratios tested. For higher replacement levels, however, when the w/c ratio exceeds 0.65 the absorptivity of concrete becomes high and the quality of concrete is characterised as poor.

Concrete strength and absorption are strongly related for PC-based concretes and hence the trends observed for equal w/c ratio matched those for equal strength (Figure 5.12). As concrete strength increases, absorption reduces and recycled aggregate content has a detrimental effect only above 50% v/v replacement level. As expected, a good correlation (R^2=0.71) was obtained between the 28-day compressive cube strength of CEM I recycled aggregate concretes and the ISAT-10 values (Figure 5.14).

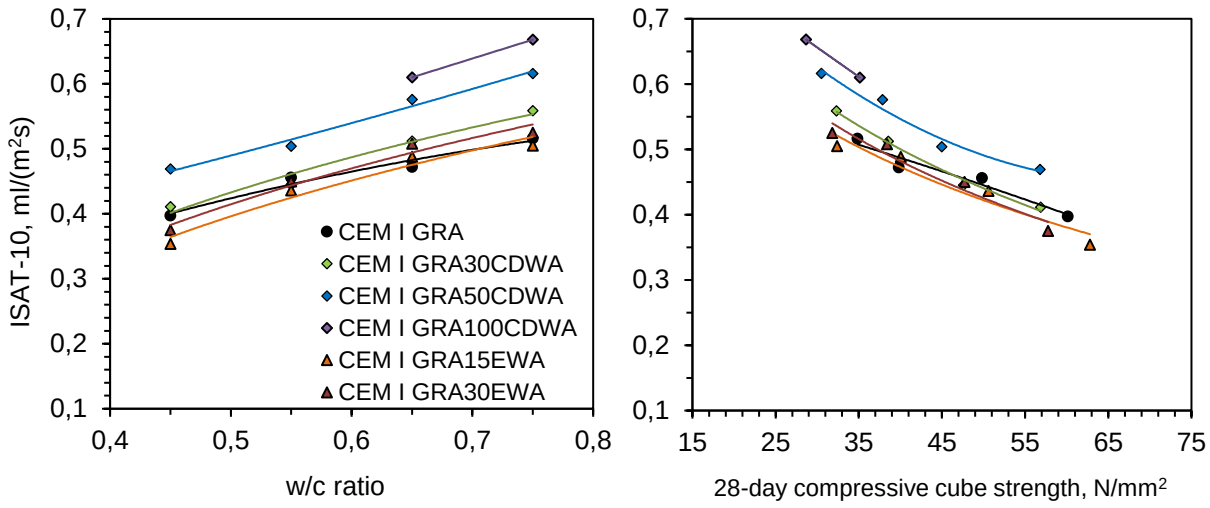


Figure 5.12: Initial surface absorption of the test CEM I recycled aggregate concretes in relation to w/c ratio and 28-day compresive cube strength

A comparison of ISAT-10 values for CEM I/FA and CEM I/GGBS recycled aggregate concretes with CEM I recycled aggregate concretes at equal w/c ratio and strength is given in Figure 5.15. Similar to what noted previously, the inclusion of 30% v/v CDWA resulted only in a minor

increase in ISAT-10 values in both cases. The results also indicated that at equal w/c ratio the FA addition resulted in an increased absorptivity compared to that of CEM I-only concrete across the whole range of w/c ratios, whereas GGBS addition had a beneficial effect. This was expected as GGBS has a higher pore blocking ability at early ages from that of FA due to its increased pozzolanicity. However, the relative differences with respect to CEM I reference concrete tended to reduce with increasing the w/c ratio.

On the contrary, at equal strength the performance of binary cement concretes is regarded as better. This improvement is a combined effect caused by (i) the higher binder content employed to accommodate strength loss and (ii) particle packing closing interconnected pores and therefore densifying the concrete. When the ISAT-10 values of binary cement recycled aggregate concretes were include in Figure 5.14 the correlation factor (R^2) was reduced to 0.56 because of the aforementioned reasons.

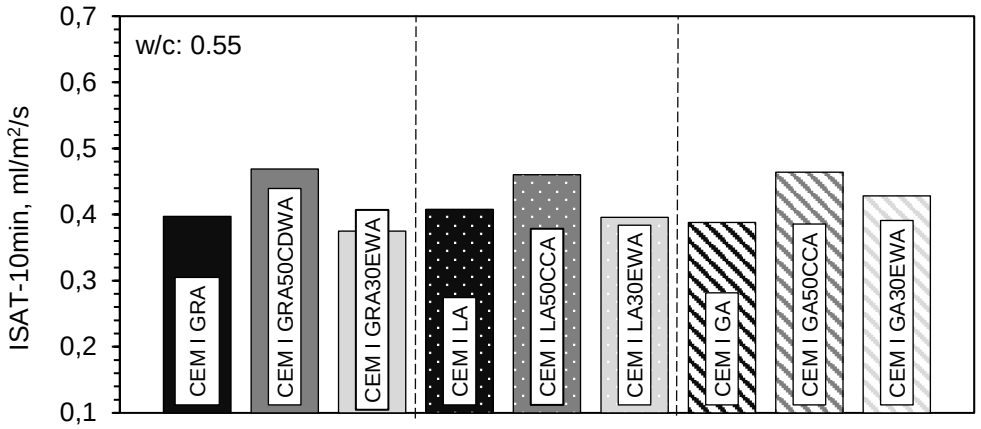


Figure 5.13: Comparison of initial surface absorption of CEM I and control recycled aggregate concretes at a w/c ratio of 0.55

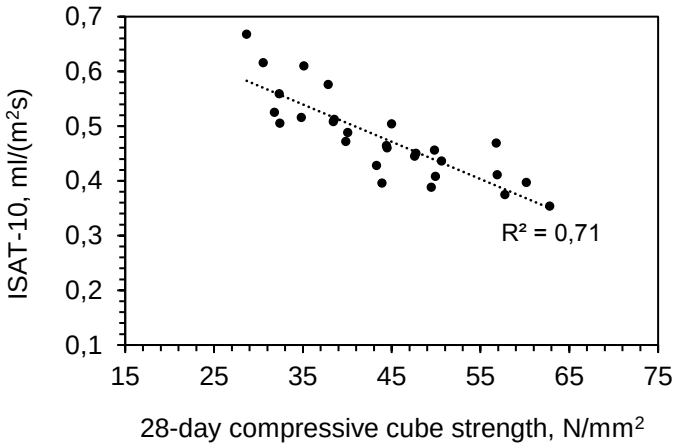


Figure 5.14: Relationship between 28-day compressive cube strength and initial surface absorption for all CEM I-based concretes

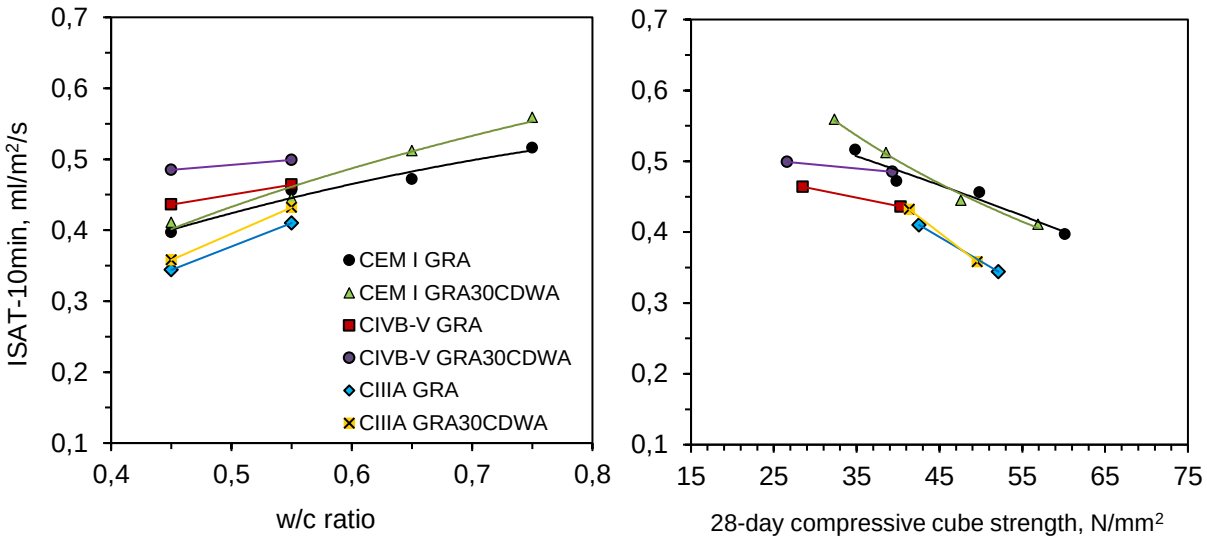


Figure 5.15: Initial surface absorption of the test CEM I/FA and CEM I/GGBS recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength

5.5 **Carbonation**

Carbonation rates of recycled aggregate concretes exposed to 4% CO_2 and measured throughout a period of 20 weeks at w/c ratios in the range of 0.45 to 0.75 are shown in Figure 5.16. The relationships obtained between the 10-week and 20-week carbonations depths and w/c ratio and 28-day compressive cube strength are given in Figure 5.17.

As expected, carbonation depths of all concretes increased with w/c ratio and exposure period. For a given w/c ratio, all recycled aggregate concretes performed similarly to reference concretes, with slight variations noted in the carbonation plots between these concretes at the various aggregate replacement levels. These variations were within the accuracy of the test i.e. ± 1.5mm. It has been shown earlier that the porosity of recycled aggregate concretes increases with aggregate replacement level and this would have potentially facilitated the diffusion of CO_2 within the concrete matrix, however, there were instances where recycled aggregate concretes displayed identical or lower carbonation depths. Similar results have been observed elsewhere (Dhir, et al., 1999; Levy and Helene, 2004) and it has been proposed that the higher cement content present in crushed concrete aggregates may provide an alkaline reserve which acts to protect the concrete surface against carbonation mechanisms. This could only stand for CDW aggregates, as EW aggregates were not containing any residual cement paste. In the latter case, this might be due to the increased content of fines reducing slightly the w/c ratio.

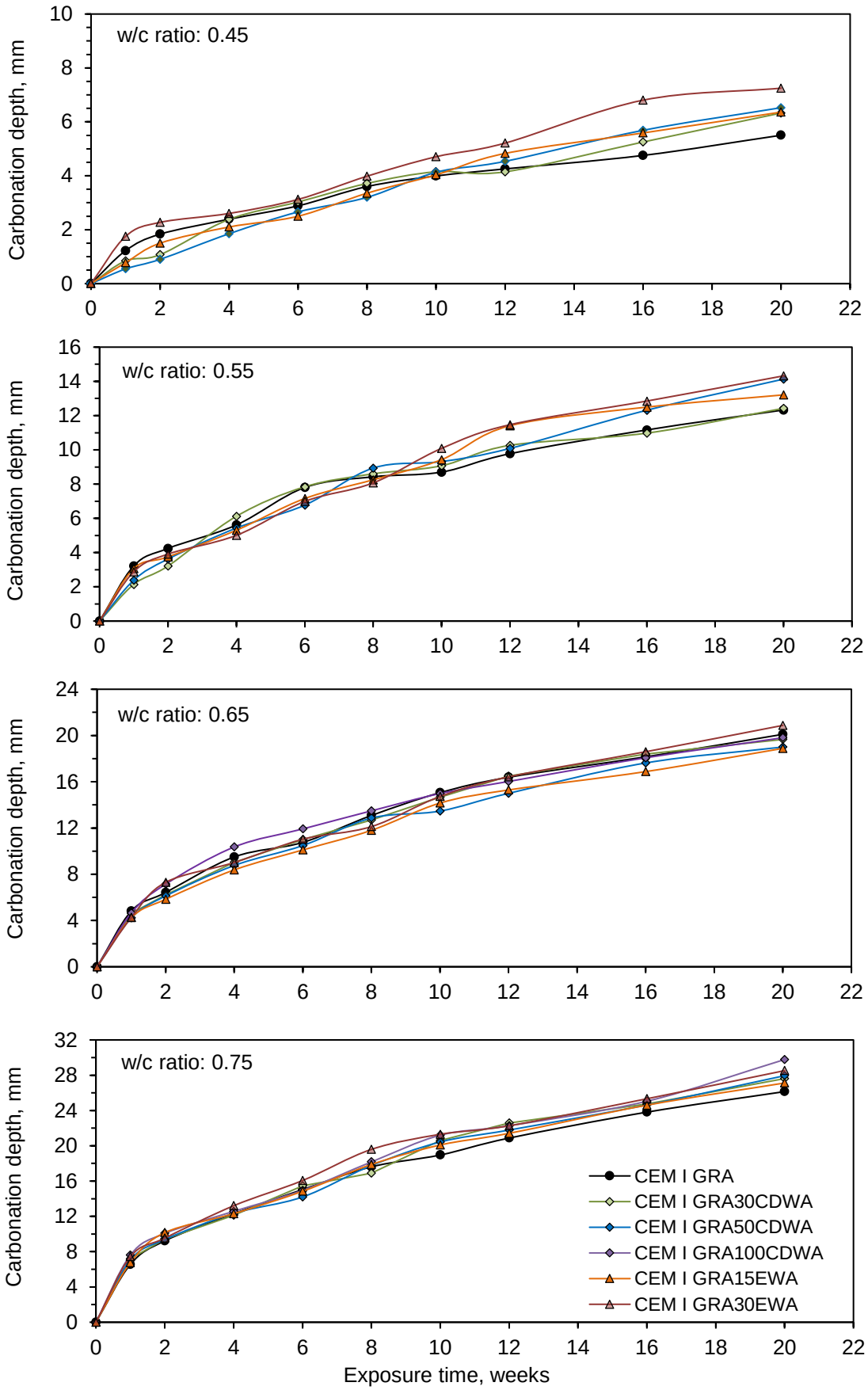


Figure 5.16: Carbonation depths of CEM I recycled aggregate concretes exposed to 4% CO_2 in relation to exposure period for various w/c ratios (carbonation depth scale adjusted for clarity)

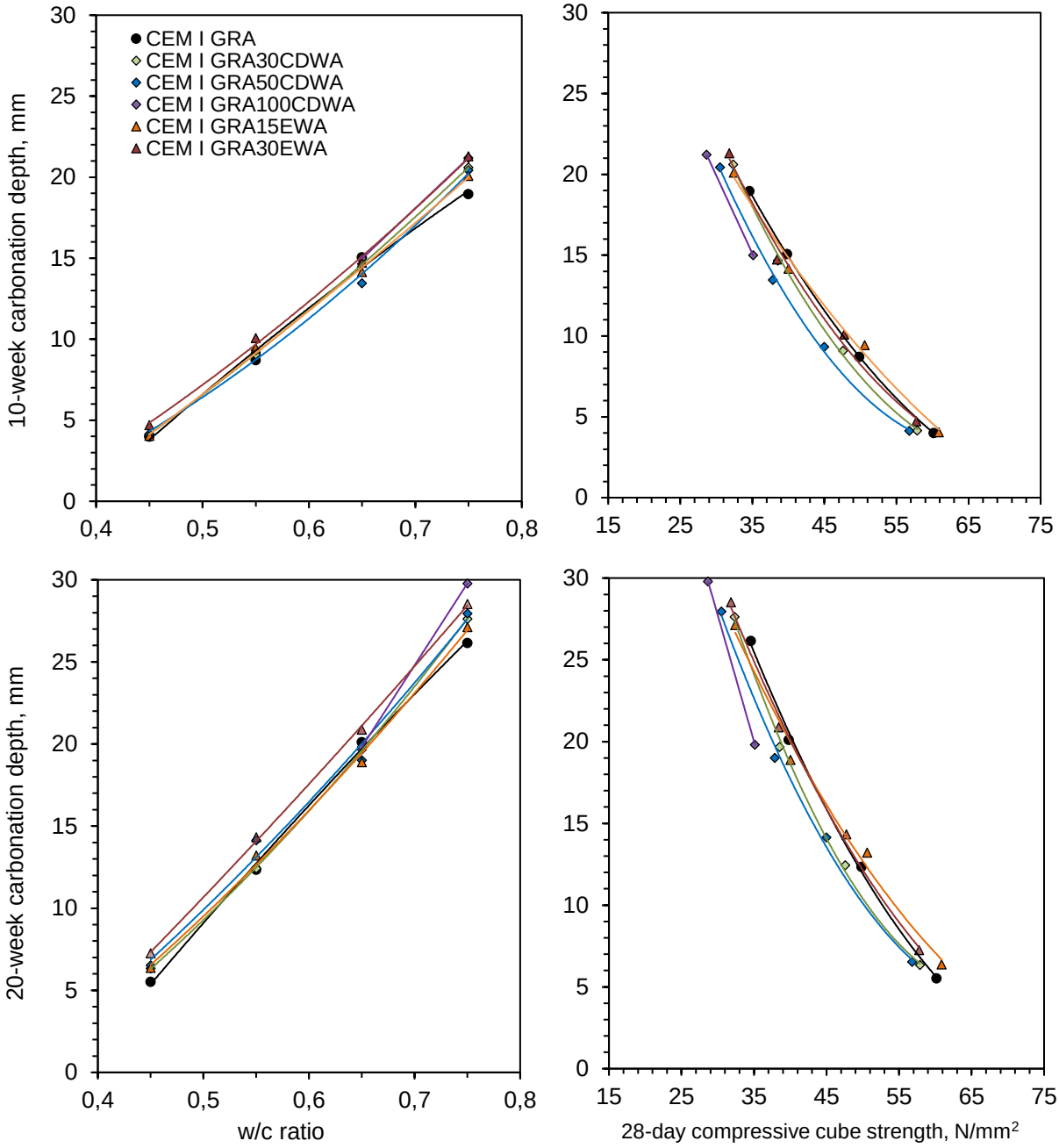


Figure 5.17: Carbonation depths of CEM I recycled aggregate concretes exposed to 4% CO_2 in relation to w/c ratio and 28-day compressive cube strength

In general, the performance of reference concretes could be regarded as slightly better by considering only the 10 and 20 week carbonation results, where the data shows a more uniform trend. At a w/c ratio of 0.45, the maximum percentage increase in the carbonation depth of recycled aggregate concretes was 25.6%. This was reduced to less than 14% at 0.75 w/c ratio even for a total aggregate replacement. This highlights once again the minimal effect of recycled aggregate properties to the performance of low grade concretes.

On an equal strength basis, recycled aggregate concretes showed slightly improved resistance to carbonation. The increased CEM I content required to achieve equivalent 28-day compressive strength reduces w/c ratio and hence diffusivity of CO_2 through interconnected pores and also provides a higher pH buffer.

Figure 5.18 compares the 10 and 20 weeks carbonation results obtained for CEM I and control recycled aggregate concretes. As with previous tests, all concretes gave comparable results at both test ages, with the differences been in the narrow range of 1-2.5mm.

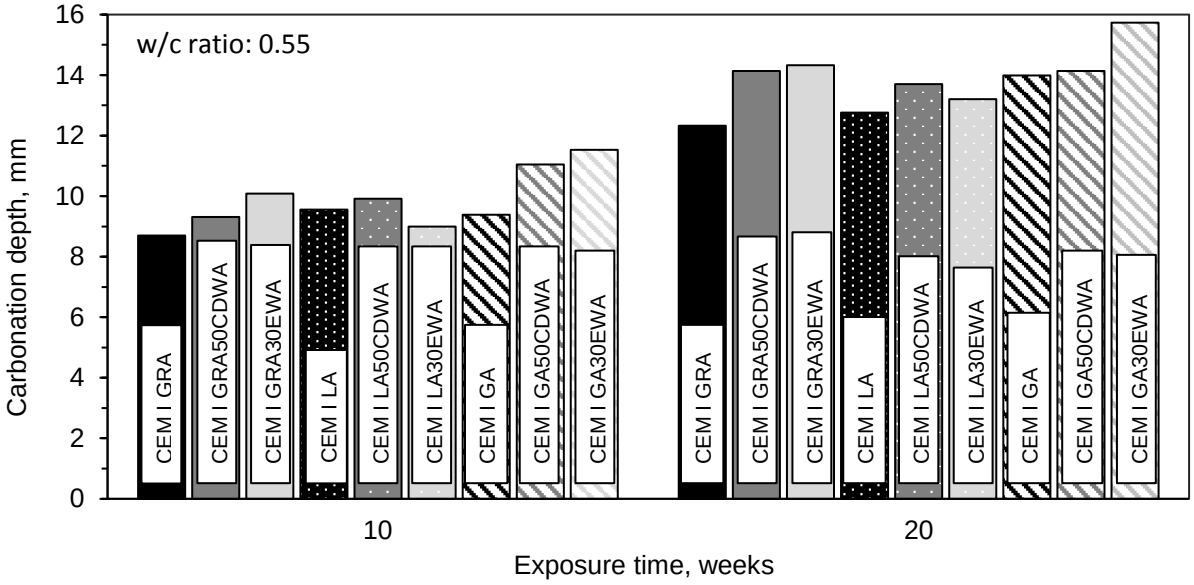


Figure 5.18. Comparison of 10-week and 20-week carbonataion depthes for the test CEM I and Control recycled aggregate concretes at a w/c ratio of 0.55

Carbonation depths in relation to exposure period for CEM I/FA and CEMI/GGBS recycled aggregate concretes are given in Figure 5.19. Comparisons with CEM I recycled aggregate concretes at equal w/c ratio and strength are shown in Figure 5.20.

In all cases, the carbonation depths of CEM I concretes were lower than the binary cement concretes at equal w/c ratio for all exposure ages. This was expected as FA and GGBS pozzolanicaly react with lime and reduce the lime available to maintain the pH level of concrete. However, the permeability of the concrete also reduces with the pozzolanic reaction. This can be seen by comparing the carbonation plots in Figure 5.19 with that of Figure 5.16. Initially, the carbonation rates of binary cement concretes were higher but after 16 weeks of exposure they began to level off. The use of 30% v/v CDWA only increased slightly the carbonation depths. The maximum difference noted between recycled aggregate and reference concretes was up to

2.5 mm at both w/c ratios and therefore the performance of these concretes is deemed to be identical.

When designing concretes for equal 28-day strength, the slower hydration rates of FA and GGBS usually means that the total binder content has to be increased to counteract for strength loss. This increase partially compensates for the reduction in available lime. Coupling this with the lower permeability leads to the result that the carbonation of binary cement concretes would not be significantly different from that of an equivalent strength CEM I-only concrete. This has been well demonstrated in literature (Papadakis, 2000; Thomas, 2013; Lye, et al., 2015). In this study all binary cement concretes have displayed a slightly higher degree of carbonation compared to CEM I concrete of the same strength class. As an accelerated method has been used to measure carbonation, the beneficial effect of FA and GGBS in relation to porosity and permeability has not been fully reflected within the short testing period. Regarding the effect of 30% v/v CDWA, the increase in binder content has offset the increase in permeability and therefore all binary cement recycled aggregate concretes have shown similar or slightly improved resistance to carbonation.

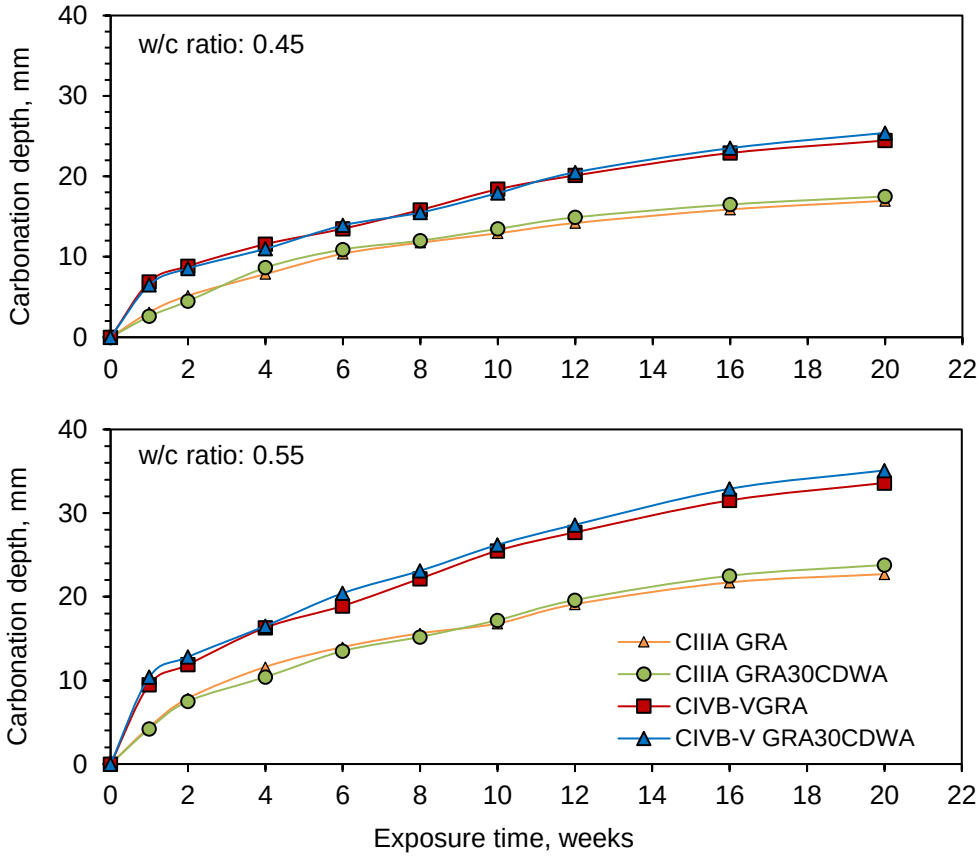


Figure 5.19: Carbonation depths of CEM I/FA and CEM I/GGBS recycled aggregate concretes exposed to 4% CO_2 in relation to exposure period for w/c ratios equal to 0.45 and 0.55

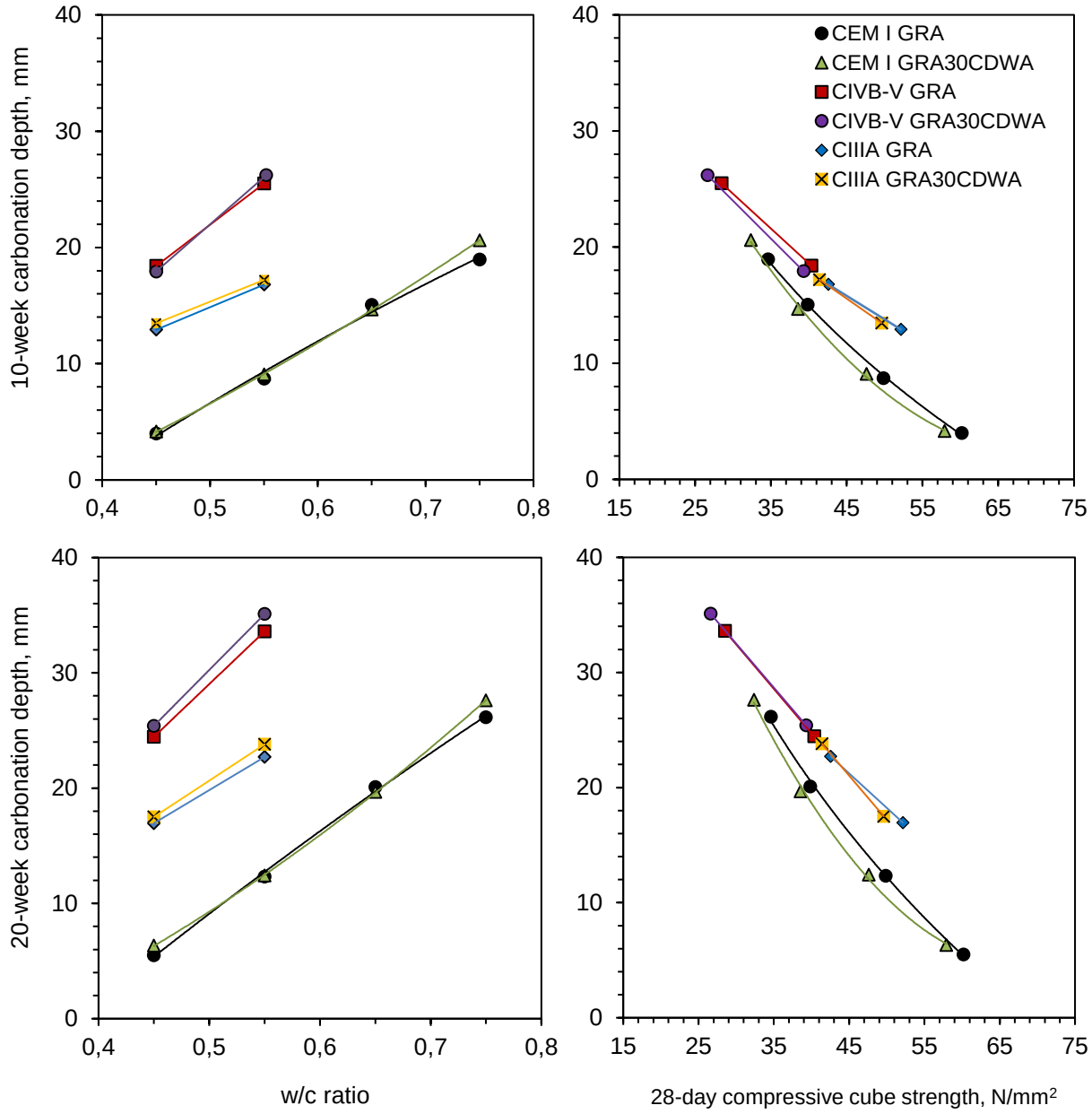


Figure 5.20: Carbonataion depths at 10 and 20 weeks of exposure week for the test CEM I/FA and CEM I/GGBS recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength

Concrete strength was always found to correlate well with degree of carbonation. This is shown in Figure 5.21 where strong relationships ($R^2 \geq 0.88$) were obtained between the 28-day compressive strength of concretes containing or not supplementary cementitious materials and carbonation depth at both test ages.

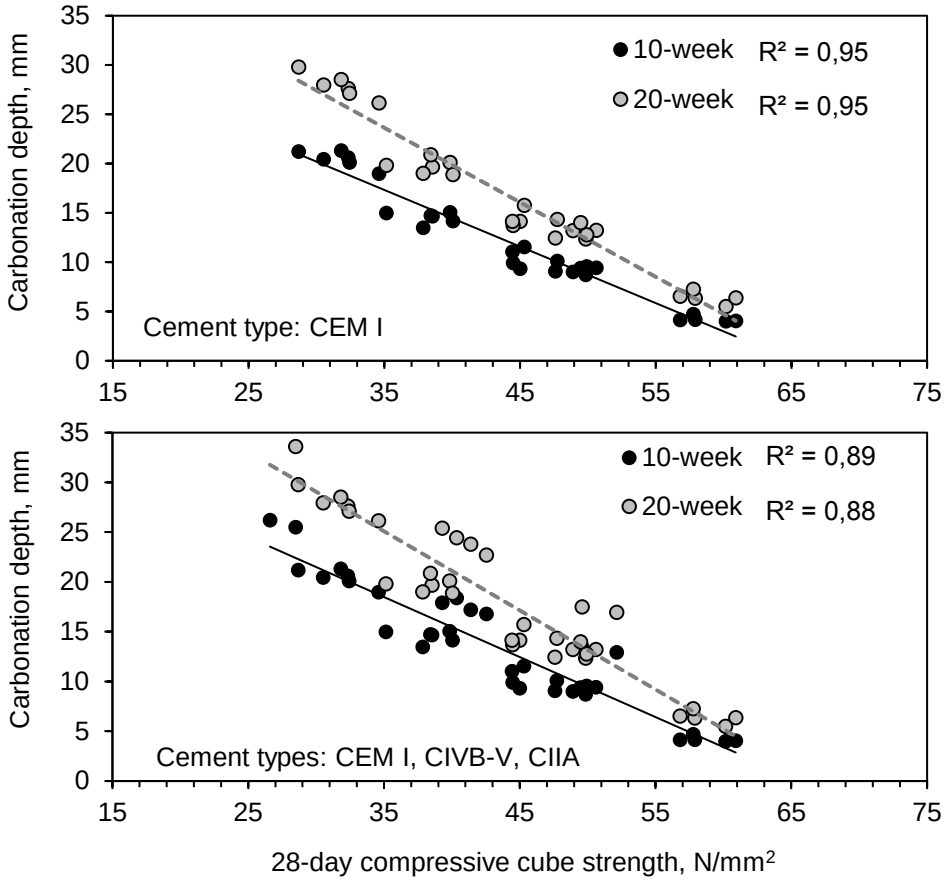


Figure 5.21: Relationship between carbonation depths of all the tested recycled aggregate concretes at 10 and 20 weeks and28-day compressive cube strength

5.6 **Chloride ingress**

The rate of ingress of external Cl^- ions into concrete largely depends on the interconnectivity of the pore network in concrete and the binding ability of the hydrated phases. Two widely known testing protocols, NT Build 492 (Nordtest 1999) and BS EN 12390 (BSI, 2015), were used in this study to measure chloride ingress and the results in relation to w/c ratio and 28-day compressive strength are plotted in Figure 5.22 and Figure 5.23.

As described in Chapter 3, NT Build 492 is an indirect test providing quickly an indication of the concrete's ability to resist chloride ion penetrability, whereas BS EN 12390 is a long duration direct test that measures the depth of diffusion of chloride ion and considers also the ability of hydrated phases to bind chloride ions during its diffusion through the concrete. In general, both test methods gave similar ranking of concrete in terms of chloride resistance. A strong correlation (R^2=0.98) was also found between these tests for CEM I-only concretes (Figure 5.24). The chloride diffusion coefficients were lower from the corresponding migration coefficients as they account for chloride binding.

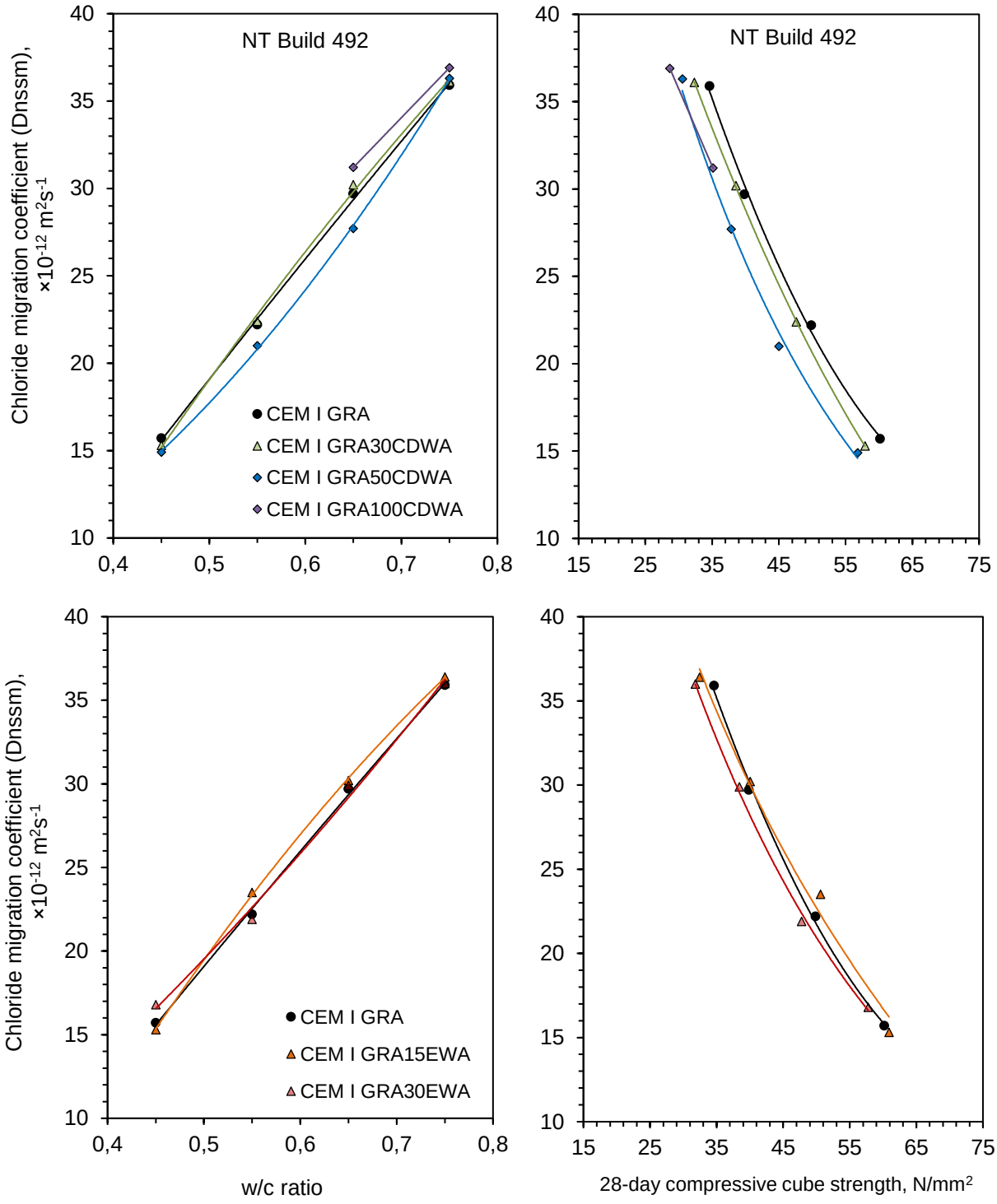


Figure 5.22: Chloride migration coefficient (Dnssm) of CEM I recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength

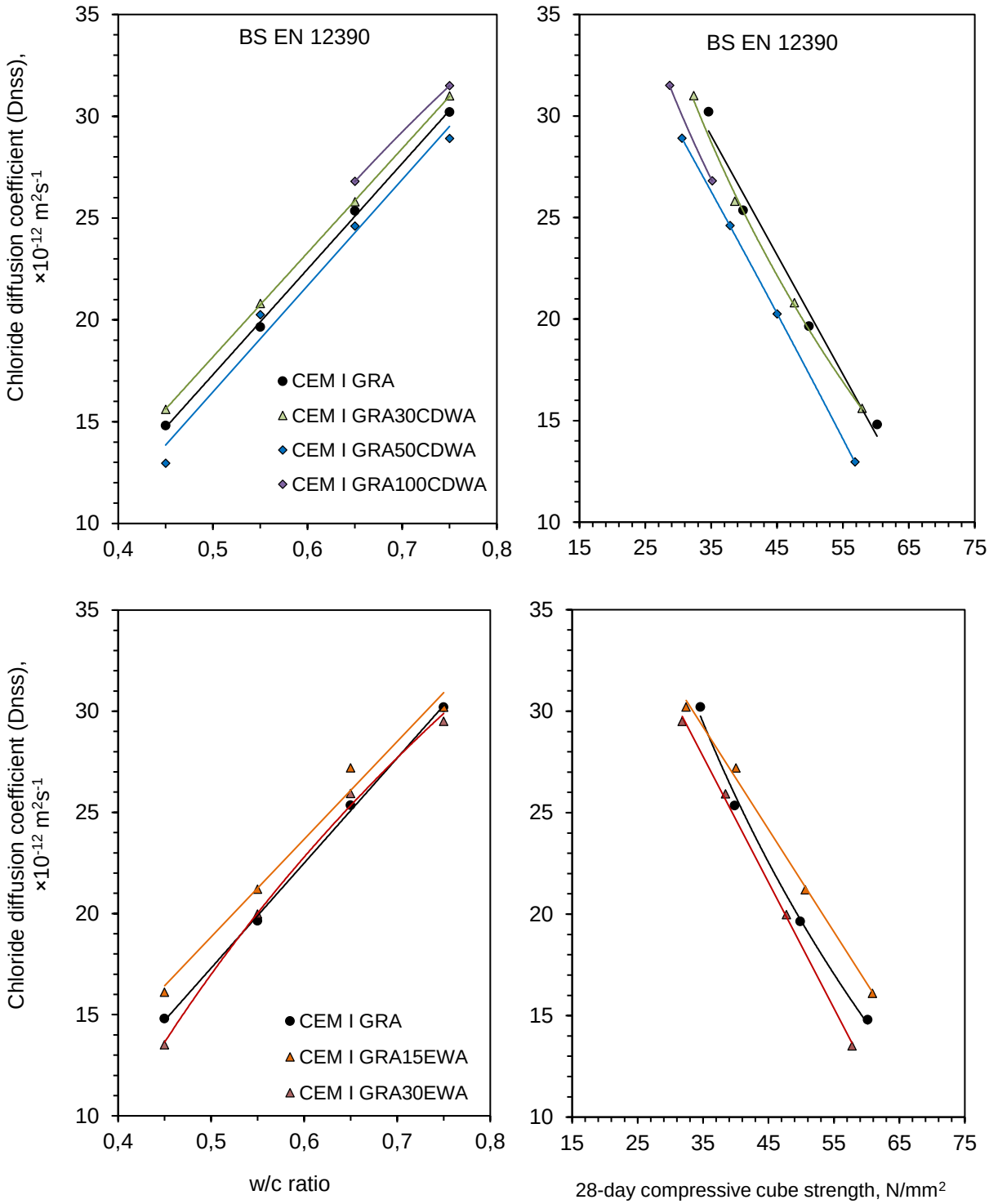


Figure 5.23: Chloride diffusion coefficient (Dnss) of CEM I recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength

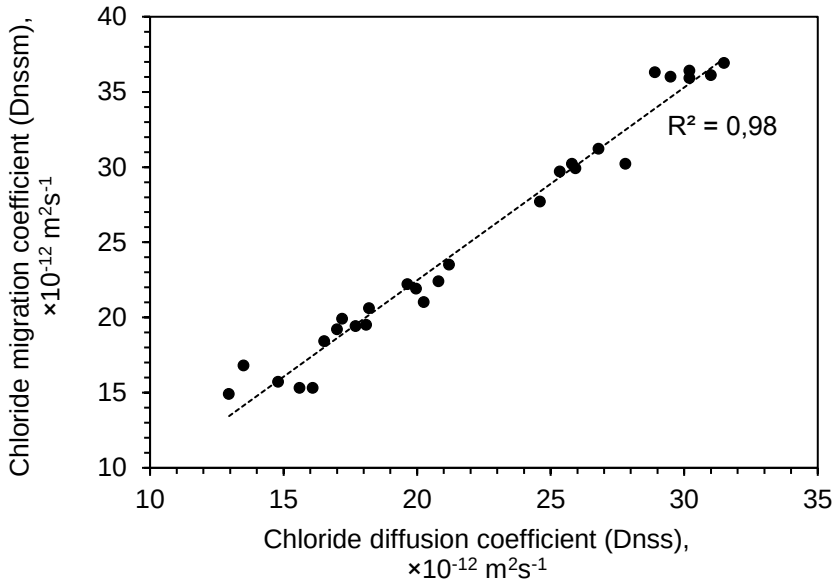


Figure 5.24: Relationship between chloride migragtion and diffusion coefficients for CEM I and control recycled aggregate concretes.

In line with established behavior the resistance to chloride ingress was reduced with increasing the w/c ratio and reducing the cube strength. The results also show that CEM I recycled aggregate concrete mixes performed as well as those containing granite aggregates, with the differences in D values between these concretes to be less than 2.0 x 10^{-12} m^2s^{-1}. Apart from CEM I GRA50CDWA mix which has constantly showed lower D values compared to reference concrete and can be considered as an outlier, it could be suggested that the use of up to 100% coarse recycled aggregates has only a minor detrimental influence on the chloride resistance of resulting concrete. This is evident in the control series mixes as well. The mixes produced with construction aggregates from Qatar displayed identical performance and slightly lower D values from those produced with UK coarse granite aggregates (Figure 5.25). The latter might be due to the slight lower density of granite aggregates or a batching error.

The minor increase in D values of CEM I recycled aggregate concretes at equal w/c ratio can be explained by the increase in porosity and interconnectivity of pore network. As Figure 5.26 shows there is a broad correlation between concrete permeability and chloride ingress, however, the strength of relationship (R^2 values) for both trend lines are moderate, suggesting a high variability between results. This is because the ISAT method gives an indirect measurement of concrete permeability. In addition to that, the permeation properties of concrete established at approx. 28 days would be slightly different at later ages, thus are not representative of how pore structure changes during the duration of the diffusion test. On the other hand, 28-day concrete

strength was always found to be a good indicator of concrete durability. Indeed, high correlation values ($R^2 > 0.87$) were obtained between concrete strength and chloride resistance for both test methods (Figure 5.27).

For the same strength class, the use of recycling aggregates up to 30% v/v has generally little or no influence in chloride ingress. At higher replacement levels, there was a reduction in migration and diffusion coefficients of approximately up to 5.0 and 3.5 x 10^{-12} m^2s^{-1}, respectively. This was the effect of reducing the w/c ratio to increase strength resulting in an improved pore system.

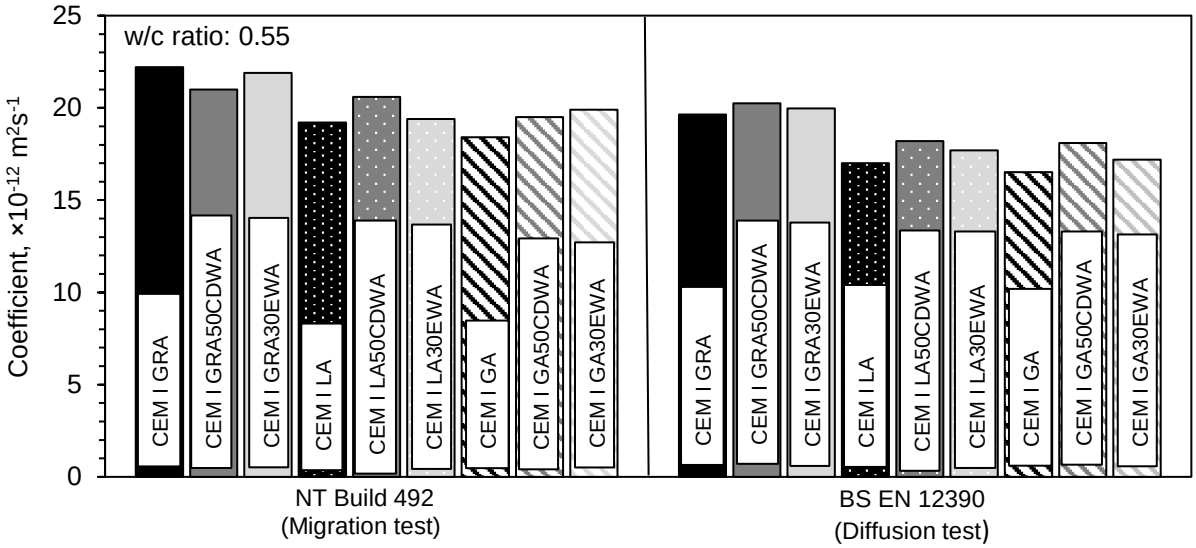


Figure 5.25: Comparison of chloride resistance of CEM I and Control recycled aggregate concretes at a w/c ratio of 0.55

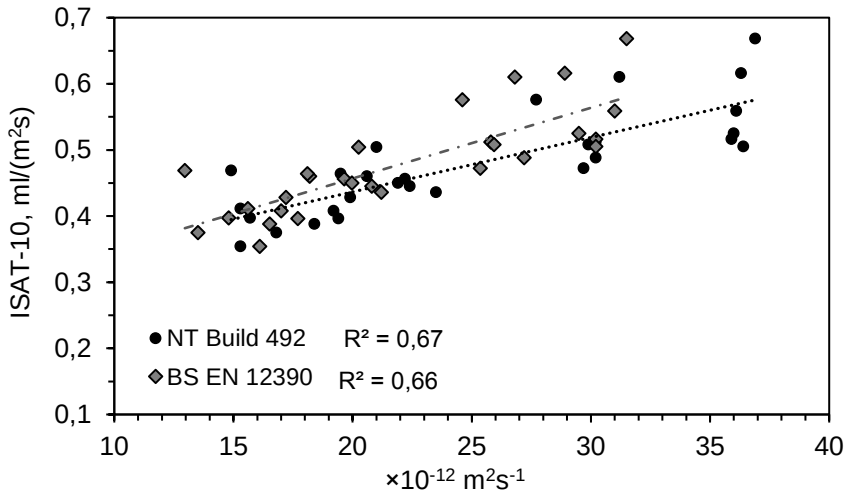


Figure 5.26: Relationship between absorptivity and chloride resistance for the test CEM I and Control recycled aggregate concretes

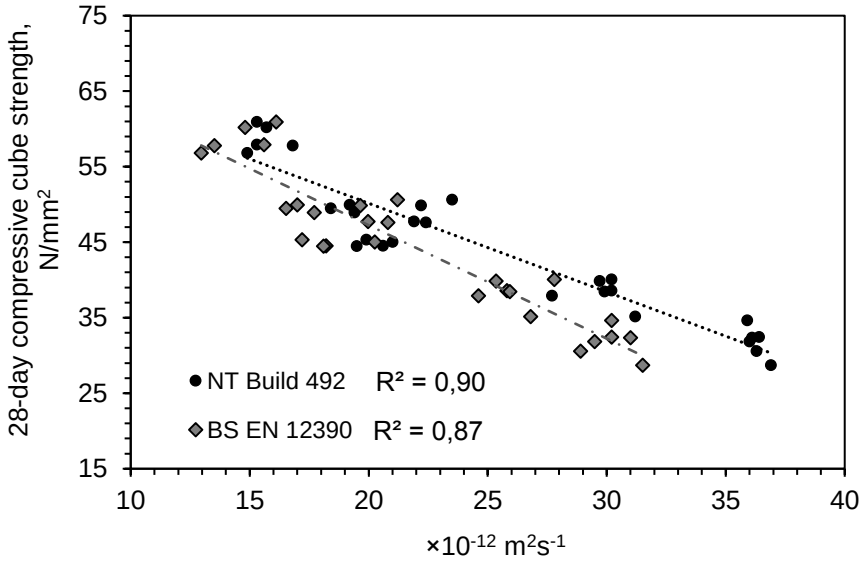


Figure 5.27: Relationship between 28-day compressive cube strength and chloride resistance for the test CEM I and Control recycled aggregate concretes

The chloride migration and diffusion results of recycled aggregate concretes containing supplementary cementitious materials are plotted in Figure 5.28 and Figure 5.29, respectively, in relation to w/c ratio and 28-day compressive strength. Similar to previous results, minimal or no effect to chloride coefficients of binary cement concretes was noted from the inclusion of 30% v/v CDWA in the mixes.

The inclusion of FA and GGBS in the mixes is a well established method to enhanced concrete resistance to chloride ingress (Dhir, et al., 1999; Papadakis, 2000, Neville, 2011; Thomas, 2013). Indeed, at equal w/c ratio the diffusion coefficients of CIVB-V and CIIIA concretes were reduced at least 65% and 80%, respectively. This reduction was a combined effect caused by (i) the chloride binding ability of FA and GGBS and (ii) particle packing closing interconnected pores and therefore densifying the concrete. The superior performance of CIIIA over CIVB-V concretes is generally attributed to the more reactive nature of GGBS providing earlier a finer pore structure and to its high alumina content increasing the chloride binding capacity of concrete. Migration test results were also in general agreement with diffusion coefficients, with the exception of CIVB-V concretes which showed slightly higher D values. This is due to the differences noted earlier regarding the age of testing and pore structure transformation with time due to pozzolanic activity. As a result the migration coefficients of CIVB-V concretes were following a similar trend to ISAT-10 results.

At an equal strength basis, migration and diffusion coefficients were in general agreement, with the latter indicating a reduction of at least 75% and 85% for CIVB-V and CIIIA concretes, respectively. This additional enhancement noted, in relation to previous percentage reductions, was due to the increased cement content contributing further towards a more impermeable matrix.

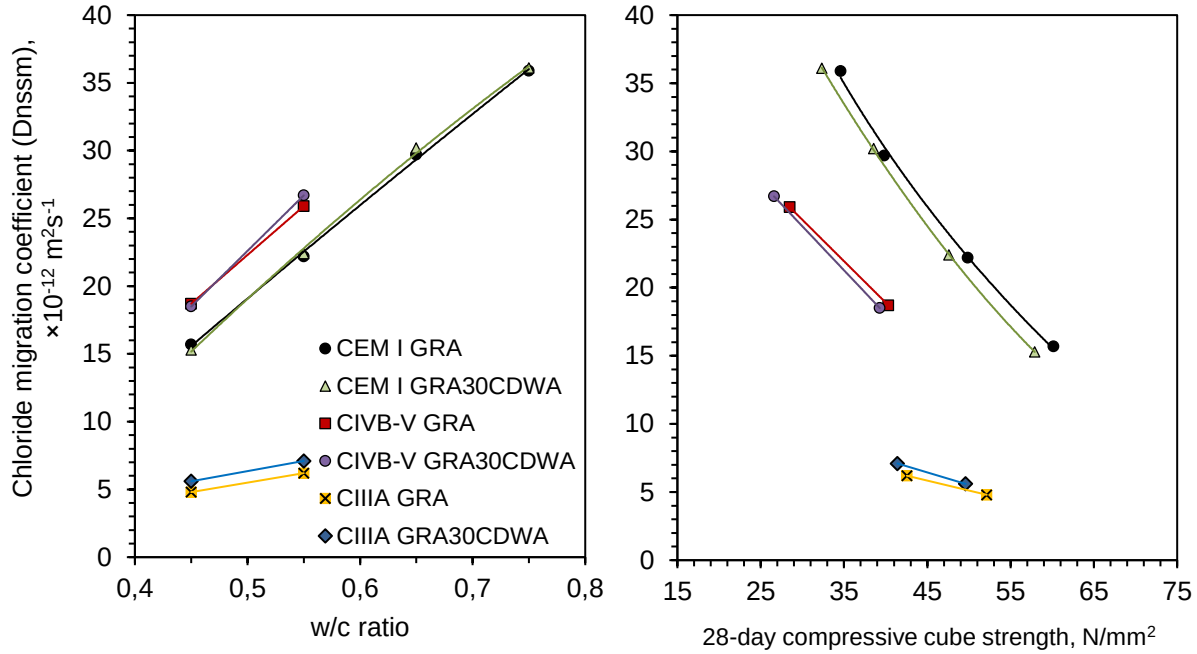


Figure 5.28: Chloride migration coefficient (Dnssm) for the test binary cement recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength

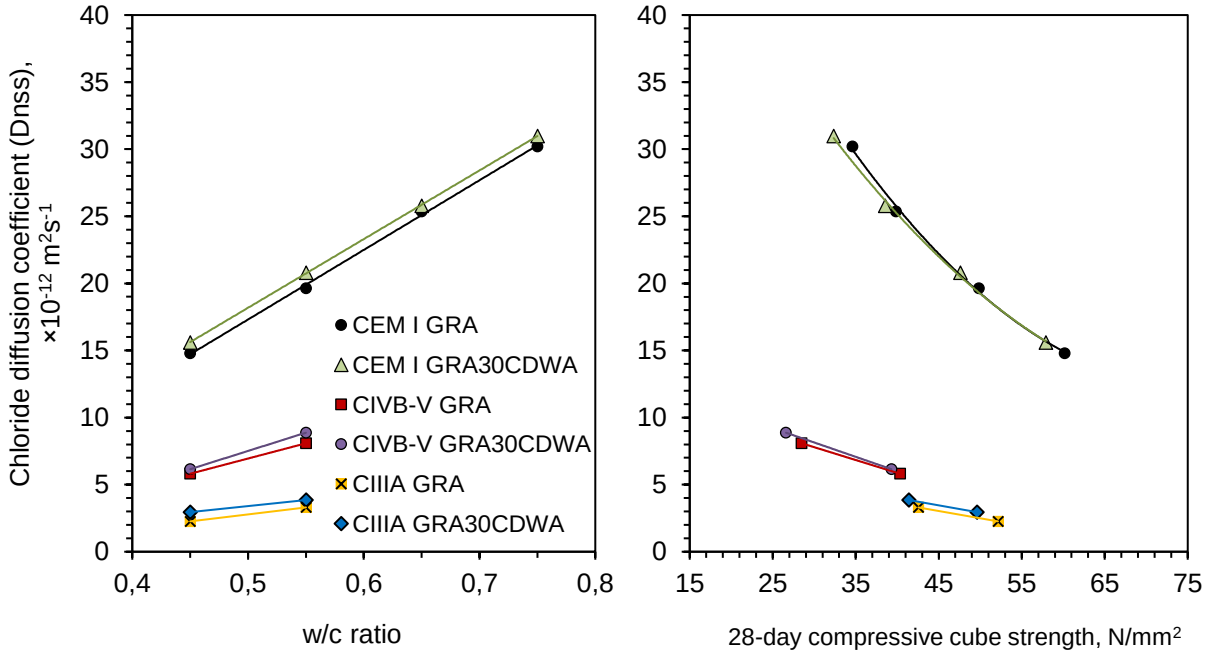


Figure 5.29: Chloride diffusion coefficient (Dnss) for the test binary cement recycled aggregate concretes in relation to w/c ratio and 28-day compressive cube strength

The compressive strength of binary cement recycled aggregate concretes was also found to correlate well with chloride ingress. Correlation values of R^2= 0.83 and R^2= 0.91 were obtained for NT Build 492 and BS EN 12390, respectively. However, when these data were incorporated within Figure 5.27, the R^2 values reduced accordingly to 0.49 and 0.15. This is because there is no link between compressive strength at 28 days and binding ability of cementitious materials or pore refinement at later ages.

5.7 **Sulfate resistance**

External sulfates penetrates through concrete body and cause expansion of the C_3A hydrate phase in Portland cement. This has been well documented by Neville (2004) and Manendez et al. (2013). It has also been suggested by Samarai (1976) that 0.1% expansion (similar to the BRE Digest 330 classification for AAR) is the limit above which significant concrete degradation does occur.

The relationship between sulfate expansion (as percentage of initial length) of CEM I recycled aggregate concretes and exposure period for the two testing conditions (fully immersed and cycling wetting and drying) and the four w/c ratios (i.e. 0.45, 0.55, 0.65 and 0.75) are shown in Figure 5.30. The corresponding mass changes are given in Figure 5.31. The 60-week sulfate expansion in relation to w/c ratio and 28-day compressive strength for both testing scenarios is shown in Figure 5.32.

The results show a general trend of increasing expansion with exposure time and w/c ratio. At lower w/c ratios the ingress rate of sulfates ions is slowed by the reduced porosity and permeability of concrete but since such mixes have a higher cementitious content (instead of a reduced water content) this would result in an increased monosulfate phase content prone to convert to expansive ettringite and, in later stages, gypsum, albeit within a denser matrix. This, however, seems not to be the case for these concretes and w/c ratio appears to be vital for controlling expansion.

At all w/c ratios, concrete samples that have been kept fully immersed into the sulfate solution have shown expansion values less than 0.1%. The increase of recycled aggregate content was only found to have a small detrimental increase at w/c ratios of 0.65 and above. At lower w/c ratios recycled aggregate concretes performed similar or better from reference concrete. It is not

clear why this happened, as it was expected the higher porosity of concretes containing elevated amounts of CDWA coupled with the increased presence of soluble sulfates originated from CDWA would have resulted in higher expansion. On the contrary, concretes containing EWA at levels of 15% and 30% v/v showed constantly the lowest expansions. This was due the underestimated water absorption of these aggregates resulting in a reduced w/c ratio, as explained previously. Concretes having a w/c ratio up to 0.65 and been exposed to a cyclic wetting and drying sulfate environment displayed expansion patterns similar to those observed for full immersion testing conditions. At 0.75 w/c ratio, however, the rate of sulfate expansion has significantly increased and all concretes showed an expansion after 60 weeks of exposure approximately two times higher than the safe expansion limit of 0.1% suggested by Samarai (1976). This suggest that the damaging effect of the physical action of cyclic crystallisation of the sulfate salts is began to be more influential at very high porosities.

CEM I and control recycled aggregate concretes have shown similar expansions at a w/c ratio of 0.55. This is illustrated in Figure 5.33. There were some differences between theses concretes but were minor. The effect of recycled aggregate content was also found to vary.

For a given strength, the use of CDWA has resulted in comparable expansions at both testing conditions and that of EWA in an improved performance. In the former case this was due to the reduction in w/c ratio to achieve equivalent strength, whereas in the latter was due to the combined effect of both reduced w/c ratio and lower permeability of these concretes.

Visually, all CEM I recycled aggregate concretes suffered from surface discoloration, softening, cracking and deterioration at edges and corners (see Figure 5.34 and Figure 5.35). The deterioration at edges is typical and due to attack in 2 plans. The whitish surface discoloration is attributed to the formation of ettringite and gypsum. Concretes exposed to alternate wetting and drying have additionally experienced minor scaling damage due to crystallisation of sulfate salts. The ongoing deterioration has also caused in a few instances some concrete spalling. This has been recorded as a loss of mass in Figure 5.31. It is difficult to say which mix had been affected most, but in general mass losses were noticed only for reference concretes or concretes containing 100% v/v CDWA and after been exposed for at least 50 weeks to a sulfate testing environment. The concretes containing recycled aggregates up to 50% v/v experienced some cracking and minor spalling but no mass loss was recorded throughout the testing period.

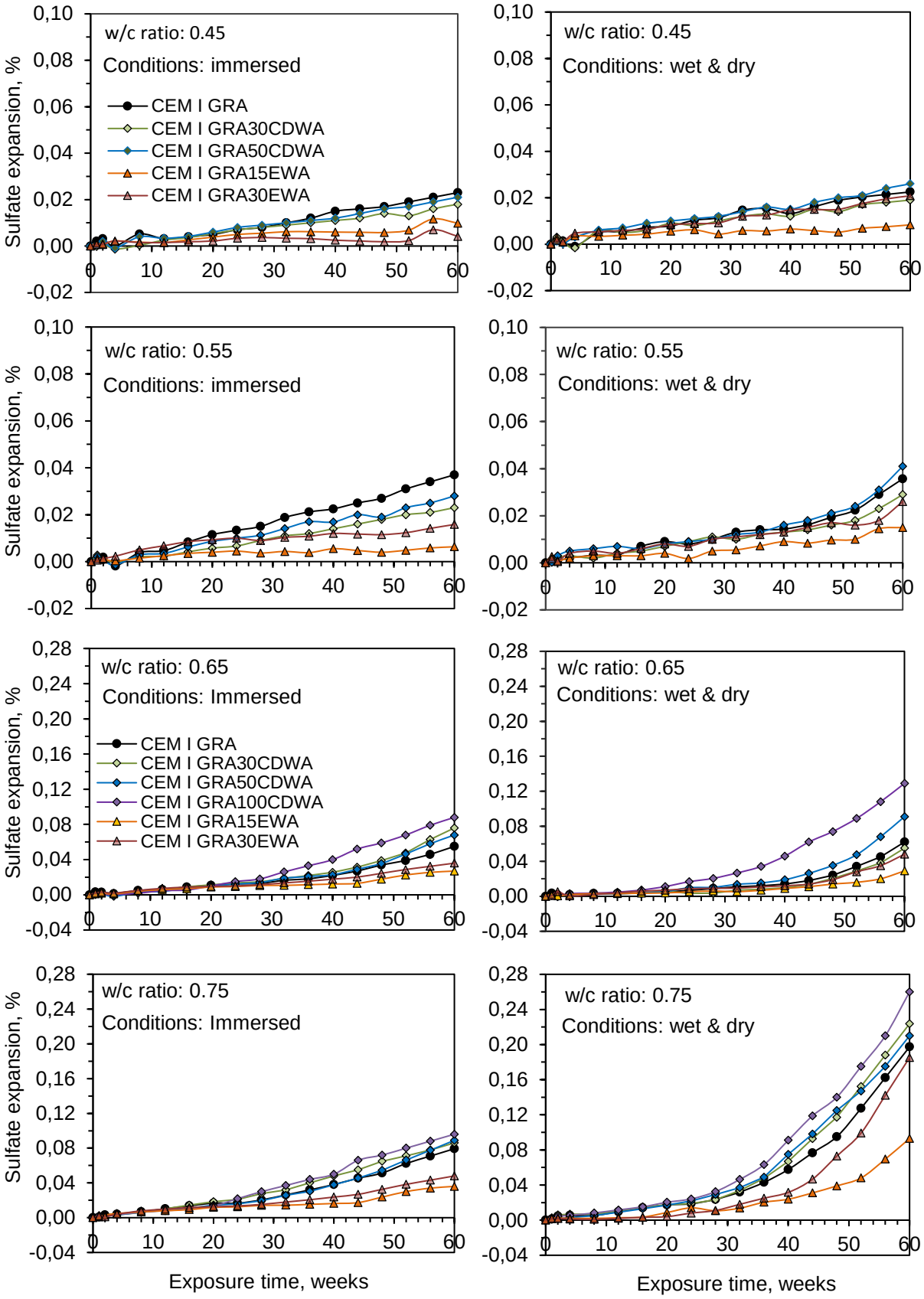


Figure 5.30: Relationship between sulfate expansion of CEM I recycled aggregate concretes and exposure time for w/c ratios equal to 0.45, 0.55, 0.65 and 0.75 (scale adjusted for clarity)

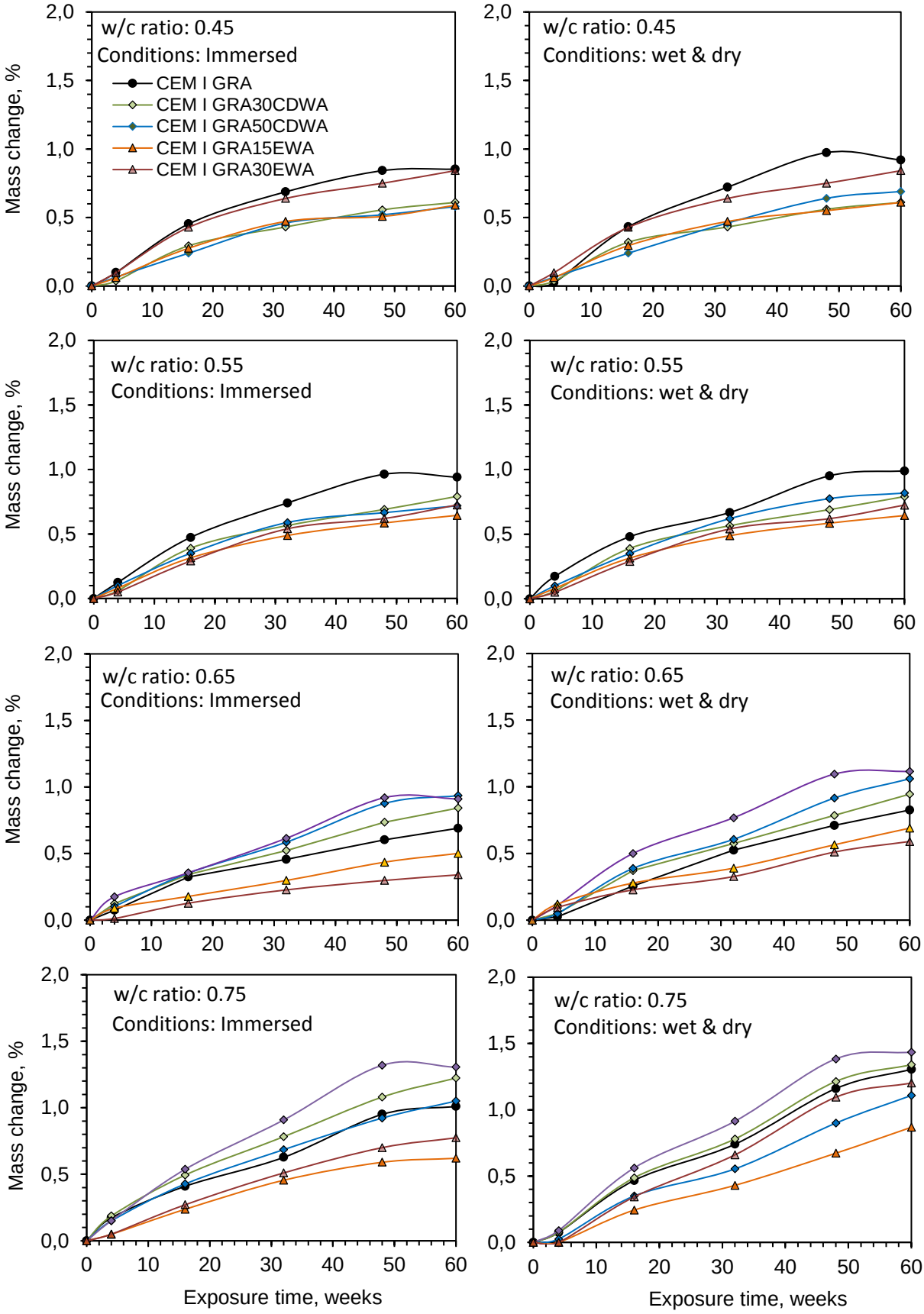


Figure 5.31: Mass change (% of initial mass) of CEM I recycled aggregate concretes exposed to sulfates in relation to exposure time for w/c ratios equal to 0.45, 0.55, 0.65 and 0.75

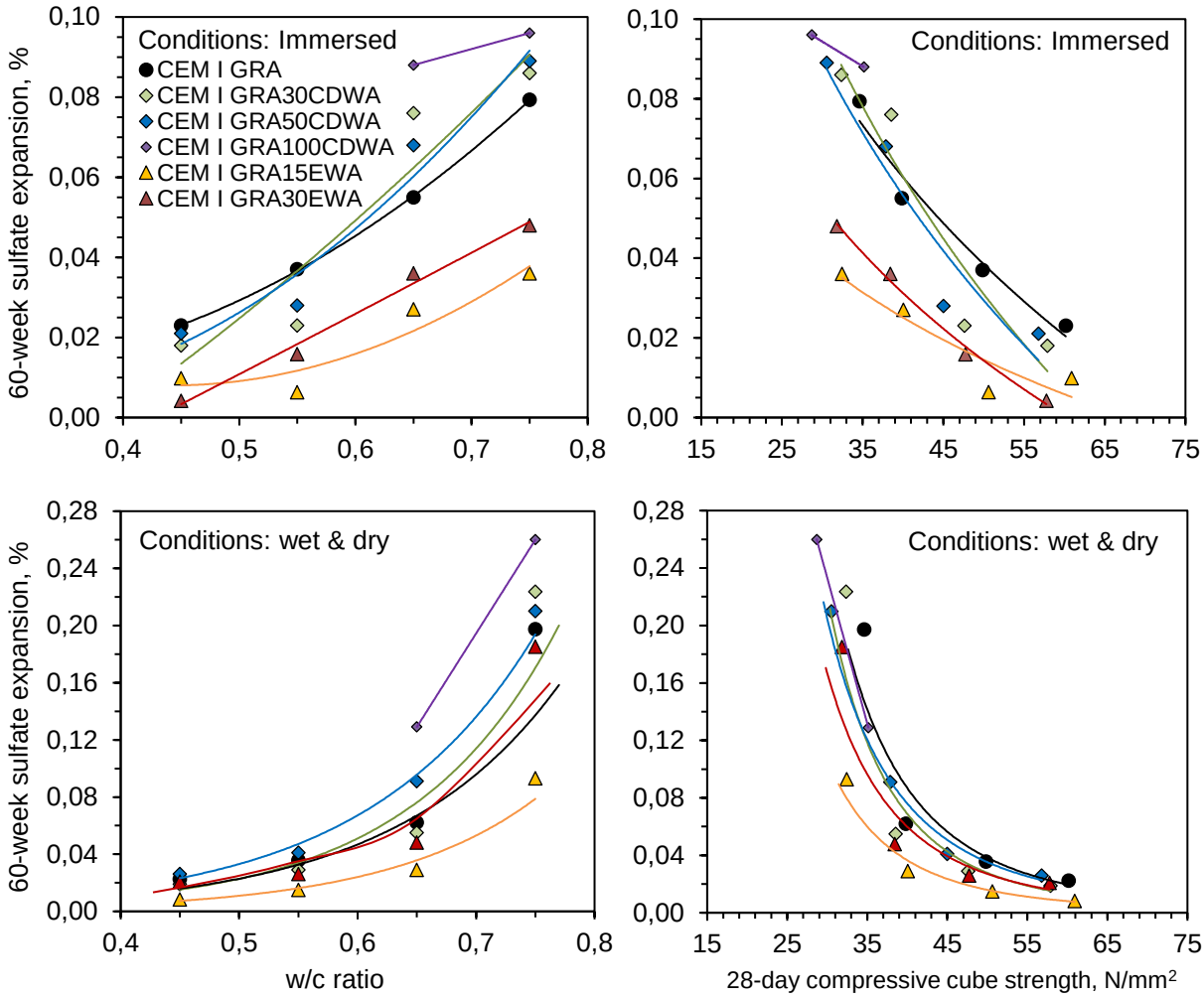


Figure 5.32: 60-week sulfate expansion for both testing conditions in relation to w/c ratio and 28-day compressive cube strength (scale adjusted for clarity)

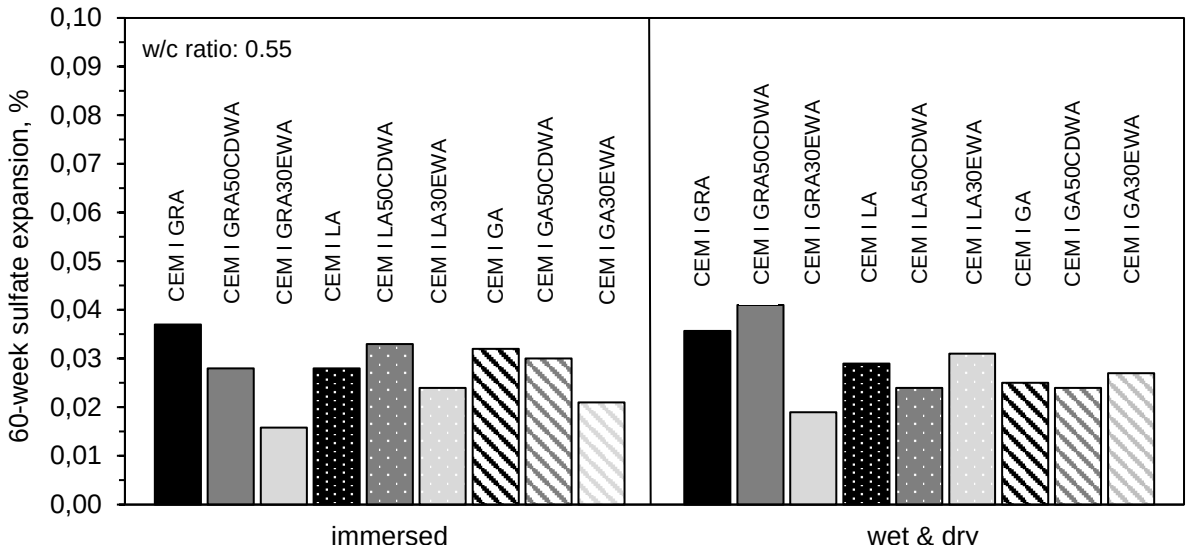


Figure 5.33: Comparison of 60-week sulfate expansion of CEM I and control recycled aggregate concretes at a w/c ratio of 0.55

Figure 5.34: Visual appearance of CEM I reference concretes after 60 weeks of exposure to sulfates

Figure 5.35: Visual appearance of CEM I recycled aggregate concretes after 60 weeks of exposure to sulfates

The sulfate expansion and mass gain of binary cement recycled aggregate concretes in relation to exposure period for the two testing conditions is given in Figure 5.36 and Figure 5.37, respectively. The corresponding 60-week sulfate expansion in relation to w/c ratio and 28-day compressive strength is shown in Figure 5.38.

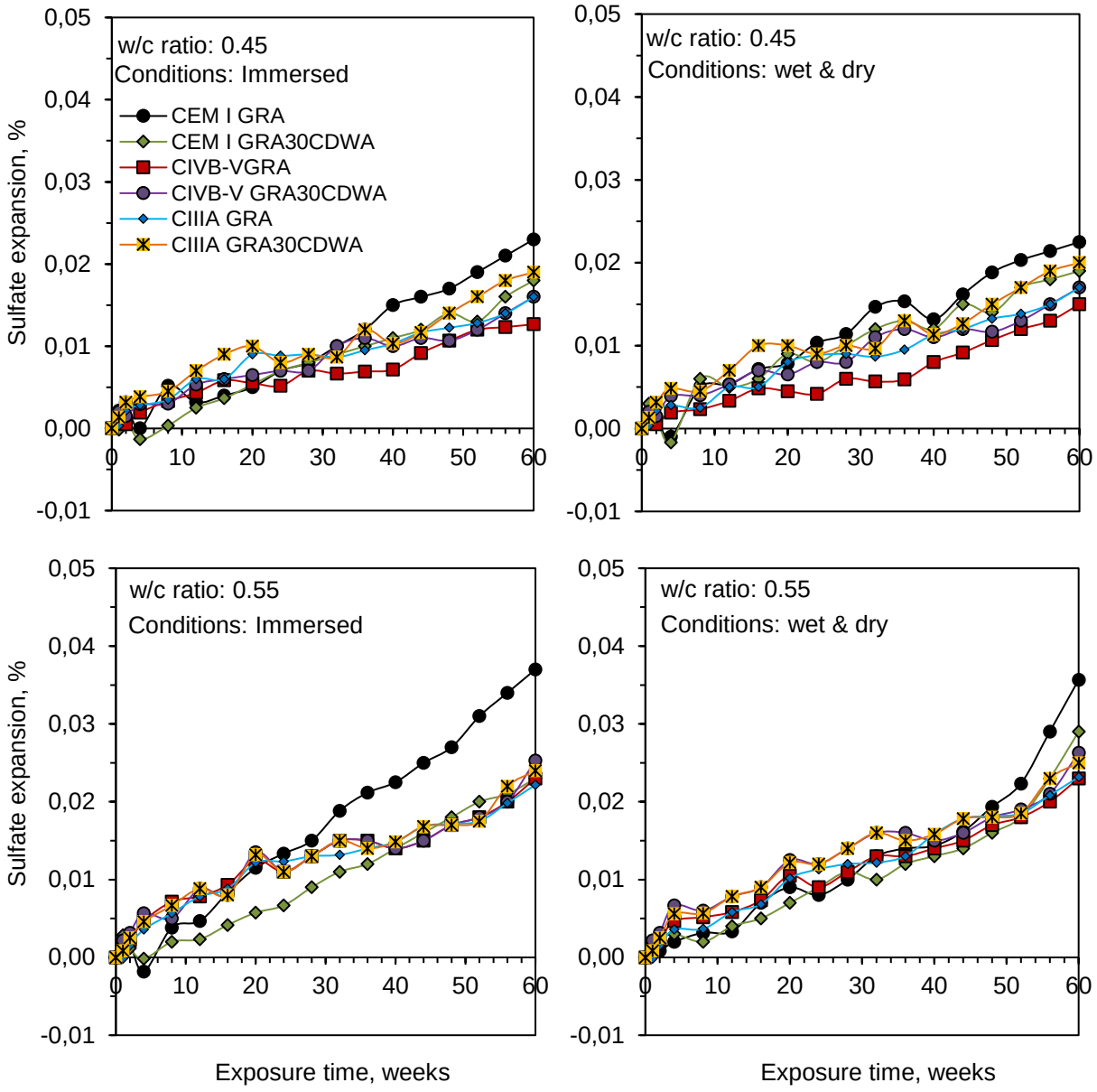


Figure 5.36: Relationship between sulfate expansion of binary cement recycled aggregate concretes and exposure time for w/c ratios equal to 0.45 and 0.55

Pozzolanic additions are typically used with PC mixes to provide a degree of protection by both diluting the amount of CAH and reducing the porosity of the concrete. Indeed, all binary cement recycled aggregate concretes showed lower expansions from reference CEM I recycled aggregate concretes at all w/c ratios. Although the CEM I/FA concretes had higher porosities both from CEM I and CEM I/GGBS concretes, the positive effect of the use of fly ash more than compensated with a reduced sulfate expansion. Similar to previous established trends, recycled aggregates at that content didn't appear to have any significant detrimental effect to sulfate expansion. Little or no difference was also noticed between the performance of these two binary cement combinations at both w/c ratios and testing scenarios.

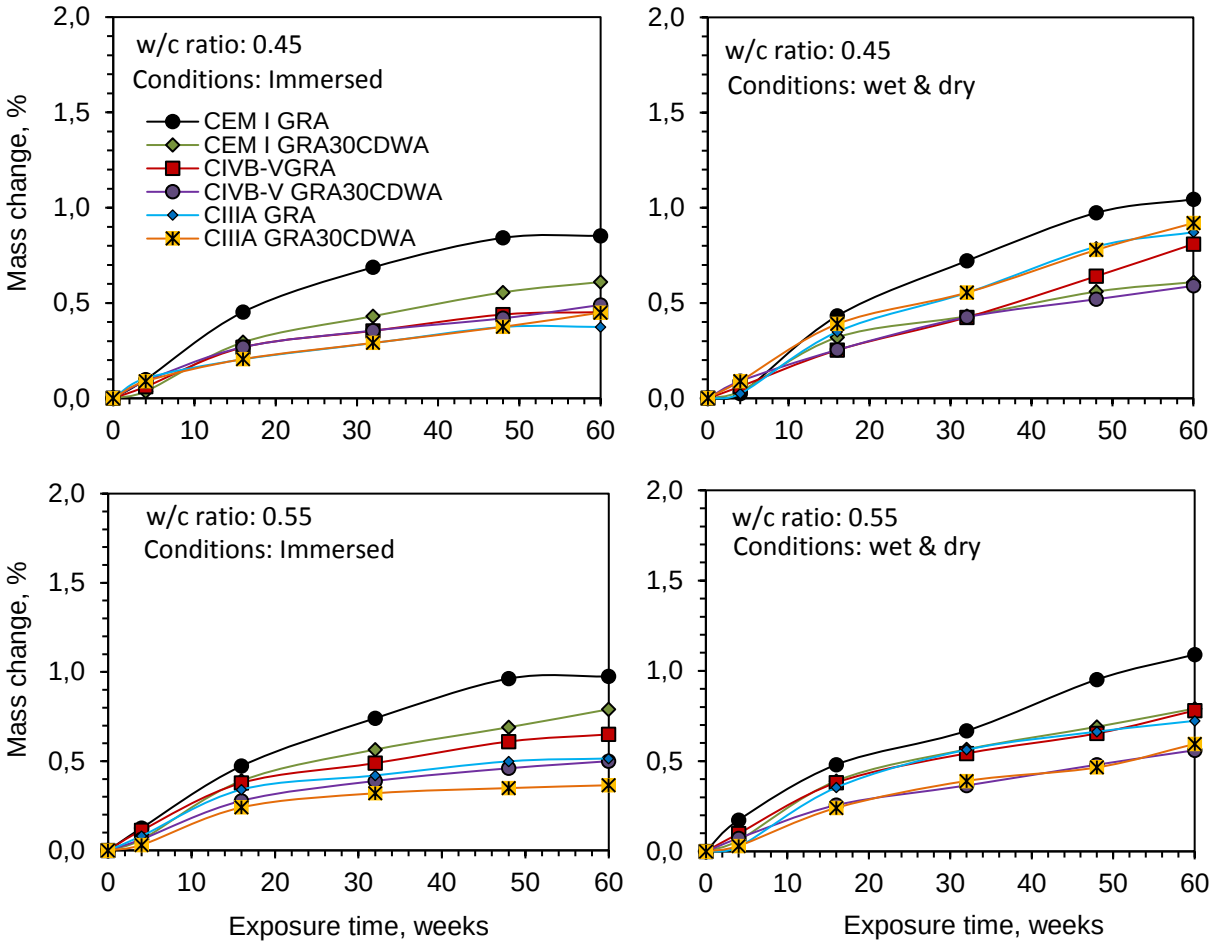


Figure 5.37: Mass change (% of initial mass) of binary cement recycled aggregate concretes exposed to sulfates in relation to exposure time for w/c ratios equal to 0.45, 0.55, 0.65 and 0.75

At all w/c ratios, CEM I/FA and CEM I/GGBS recycled aggregate had relatively lower mass gain, possibly due to fewer reaction products forming in these cementitious systems, i.e. reduced gypsum and ettringite formation. Based on visual observations (Figure 5.39), binary cement concretes gave better performance than reference CEM I concretes, with no mass losses. Whitish surface discolouration was apparent to all samples and in a few cases, particularly under cyclic wetting and drying, minor cracking and spalling was noted close to the edges.

After 60 weeks of continuous exposure to sulfates there was no difference between the high and lower w/c ratio mixes, which suggests that CAH dilution by the pozzolanic addition was the major effect in these concretes. Consequently, when the expansions of the test concretes compared on an equal strength basis, binary cement concretes outperform the reference CEM I concrete at both testing scenarios.

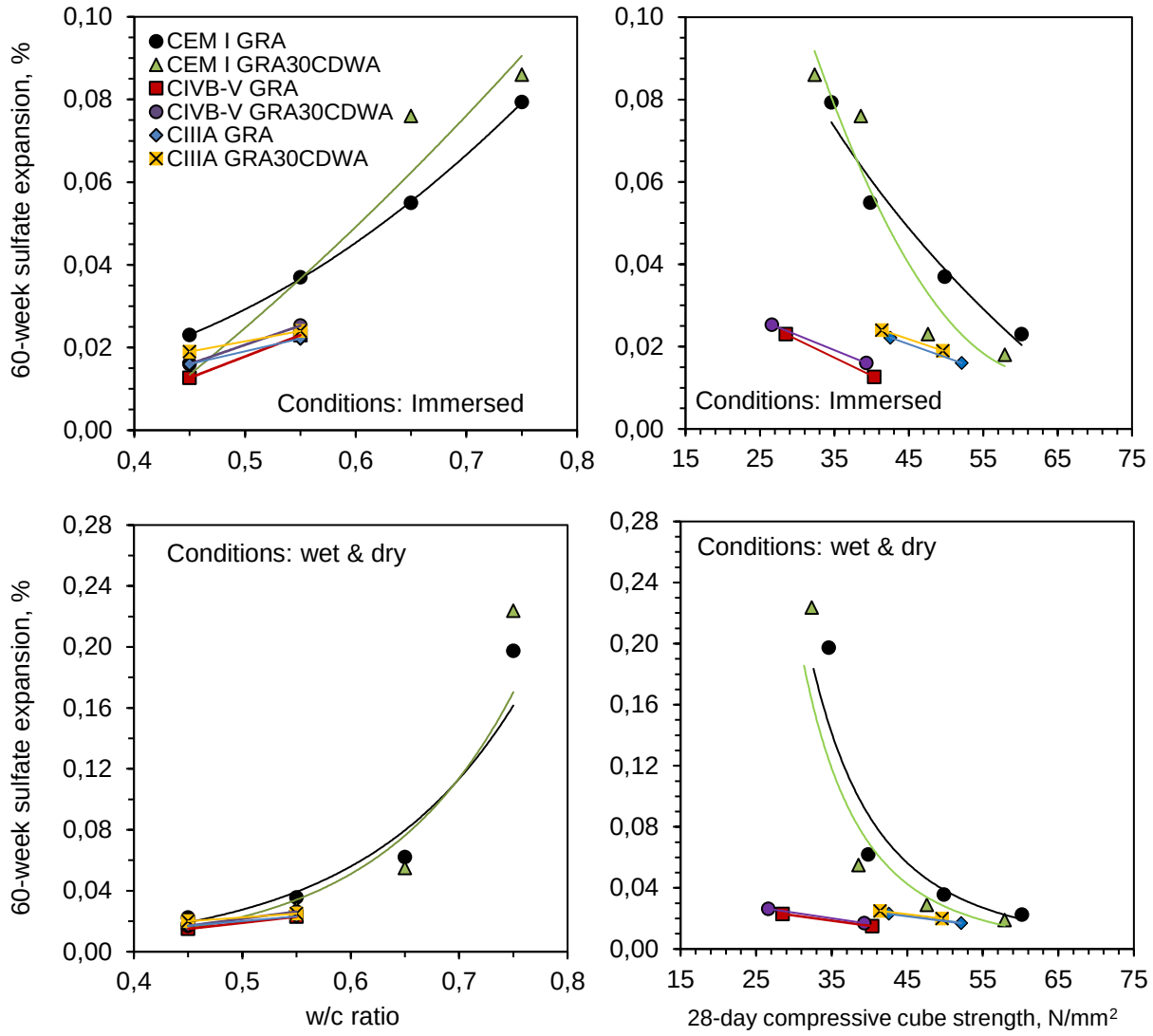


Figure 5.38: 60-week sulfate expansion of binary cement recycled aggregate concretes for both testing conditions in relation to w/c ratio and 28-day compressive cube strength (scale adjusted for clarity)

Figure 5.39: Visual appearance of binary cement concretes after 60 weeks of exposure to sulfates

5.8 **Conclusions**

5.8.1 Fresh properties

As expected, the plastic density of all CEM I-based recycled aggregate concretes was found to decrease with replacement level, owing to the lower densities of recycled aggregates. For a single w/c ratio, the incorporation of elevated amounts of coarse CDWA was generally found to improve consistence and no increase in superplasticizer dose was required. This was the effect of the more spherical shape of CDWA particles compared to UK granite aggregates, as a similar behaviour was not observed when CDWA replaced gabbro or limestone aggregates having all comparable flakiness index values. On the contrary, coarse EWA reduced drastically the consistence of the concrete, even at small additions, due to the increased presence of fines and a higher superplasticizer dose was required. Additional screening or washing to remove the excess of fines is deemed to be vital for the use of the material in structural concrete.

Additions of FA and GGBS at mass replacement levels of 40% and 50% by mass of cement, respectively, were found to reduce the superplasticizer requirements of the mix for the same consistence class. This is mainly due to the low specific surface area and smooth round particles of these materials.

5.8.2 Engineering properties

Test results showed that CDWA and EWA up to a 30% v/v replacement level had no or little detrimental effect on the compressive strength of concrete, but thereafter this reduces with increase in recycled aggregate content. This was observed consistently for concrete mixes tested at 7, 28, 90 and 180 days. A similar relationship was obtained between 28-day flexural strength and recycled aggregate content. No difference was noted in the strength performance of concrete containing CDWA or EWA at the same level. For the same w/c ratio, FA and GGBS additions also resulted in lower compressive strength at all testing ages, with the CEM I/GGBS concretes outperform the CEM I/FA concretes. To take account of the effects of recycled aggregates and pozzolanic additions on compressive strength, a simple adjustment of w/c ratio is required. For concretes containing recycled aggregates up to 50% and 100% w/w the w/c ratio has to be reduced by approx. 0.5 and 1.0, respectively, whereas for pozzolanic additions the reduction was found in the range of 1.0 to 2.0. The latter value corresponds to mixes containing FA and it is due to the lower pozzolanic reactivity of fly ash.

The effect of recycled aggregates on drying shrinkage of concrete was found to increase with replacement level and w/c ratio. At both equal w/c ratio and strength, the incorporation of 50% v/v CDWA had a slight detrimental effect to drying shrinkage which was reduced with time, whereas EWA at 30% v/v replacement level resulted in comparable or slightly lower drying shrinkage values.

Examination of water absorption characteristics of recycled aggregate concretes showed that replacement levels up to 30% had no or little detrimental effect and thereafter ISAT-10 values increase with recycled aggregate content. The trend was similar to that observed for strength data and in fact a good correlation was found between 28-day compressive strength and ISAT-10 values. At replacement levels above 50% v/v, the reduction in w/c ratio required to achieve equivalent strength couldn't compensate the increase in water absorptivity of concrete due to lower porosity of CDWA and hence these concretes continued to display higher absorption values from reference ones.

For the same w/c ratio, the addition of FA has slightly increased water absorptivity of the resulting concrete, while GGBS addition found to restrict that. At equal strength, however, both binary cement concretes showed reduced absorptivity due to the combined effect of higher

binder content employed to accommodate strength loss and particle packing closing interconnected pores.

5.8.3 Durability properties

The behaviour of concretes followed established trends, with carbonation depth increasing with w/c ratio and exposure time. Regardless the recycled aggregate type and content, all recycled aggregate concretes performed similarly to reference concretes for all w/c ratios, with differences between these lying within the accuracy of the test (±1.5mm, BS EN 1881-210, 2013). A slight increase in carbonation depths was only noticed after 10 weeks of exposure to 4% CO_2. For a given strength, recycled aggregate concretes showed comparable or slightly improved resistance to carbonation as a result of the reduced w/c ratio restricting diffusivity of CO_2 and higher cement content providing an increased alkaline reserve. CEM I/FA and CEM I/GGBS concretes initially experienced higher carbonation depths at all w/c ratios but the rate of carbonation was reduced with time. At equal strength, the increase in binder content has partially compensated the reduction in available lime, but still the resistance of these concretes was lower than the reference CEM I concrete.

Resistance to chloride ingress, measured both by migration and diffusion testing protocols, was reduced with increasing the w/c ratio and increasing the cube strength. Recycled aggregate concretes performed as well as reference concretes at equal w/c ratio, with only a minor increase in resulting coefficients towards higher replacement levels. For the same strength class, the use of recycling aggregates up to 30% v/v has generally little or no influence in chloride ingress, whereas at higher substitution levels the resistance was improved due the w/c ratio effect. The inclusion of FA and GGBS in the mixes has resulted in a superior performance for the same w/c ratio, reducing chloride ingress by approximately 60% and 80%, respectively. This reduction was a combined effect caused by (i) the chloride binding ability of FA and GGBS and (ii) particle packing closing interconnected pores. At equal strength, the chloride resistance in CEM I/FA and CEM I/GGBS concretes was further improved by the reduced w/c ratio.

Similar to other properties, the sulfate resistance of recycled aggregate concrete was reduced with increasing w/c ratio and exposure time. Recycled aggregate at all replacement levels concretes gave comparable expansions with reference concretes at w/c ratios up to 0.55 and slightly higher at 0.65 and above. Exposure to a cyclic wetting and drying sulfate environment was found to influence and increase expansion only to 0.75 w/c ratio concretes. At equal strength

however, the reduction in w/c ratio has offset any detrimental effect and all recycled aggregate similar or improved sulfate resistance. CEM I/FA and CEM I/GGBS concretes performed better than CEM I concretes at all w/c ratios and for a given strength. The expansions were generally low, with only minor differences observed between these binary cement concretes. Visually, CEM I concretes experienced discolouration and deterioration (cracking and spalling), whereas binary cement concretes were mainly unaffected.

CHAPTER 6: CALCIUM SULFOALUMINATE BELITE CONCRETE

This Chapter investigates the fresh, engineering and durability properties of concrete made with commercially manufactured CSAB cement. The concrete tested are those outlined in Section 3.5.1 and tested as described in Section 3.5.4. The concrete properties were compared at both equal w/c ratio and strength. The results were related to the findings of paste study and correlations were also made between various properties to establish the behaviour of the material. Finally, the use of various material combinations is discussed in a performance related approach along with the achieved CO_2 savings.

6.1 Fresh properties of concrete

The fresh properties of CSABg recycled aggregate concretes are given in Table 6.1 and those of CSABg/FA and CSABg/GGBS concretes in Table 6.2. The plastic density measurements gave general agreement with the theoretical density (± 20 kg/m^3) for all concrete mixes and this provides confidence that the concretes were batched correctly.

The SP dose was adjusted for each w/c ratio to achieve the required S3 consistence class (i.e. 100 to 150mm slump) and then kept constant for mixes containing CDWA to establish their effect. In line with the established behaviour in Section 5.1, the workability of concrete was found to improve slightly with the addition of 50% v/v CDWA. For the same w/c ratio and SP dose, the inclusion of FA and GGBS at 15% w/w additions resulted in slump values (at 5 and 45 min) similar to those obtained for the reference CSABg GRA concrete. This has been noted previously (Zivica, 2000; Zivica, 2001; Ioannou, et al., 2014) for CSA-based pastes containing FA or GGBS and may be due to pozzolana containing small amounts of calcium sulfate.

Although all CSABg-based concretes initially met the target of S3 class, they experienced a substantial numerical loss of slump within the next 40 minutes. This was greater for the lower w/c ratio mixes. Remixing of concrete every 15 minutes didn't appear to restore the initial consistence, as it usually happens with PC-based concretes. A similar trend was noted by Dachtar (2004). The reduction in fluidity of CSABg concretes is due to two main issues. Firstly, the rapid hydration of ye'elimite phase and the formation of a network of ettringite crystals, as shown by setting time measurement on CSAB/gyspum pastes in Section 4.2.2, and secondly the rapid 'consumption' of free water, as 32 H_2O molecules are bound according to the chemical

formula (Glasser and Zhang, 2001; Zhang and Glasser, 2002). It should be highlighted that no retarding agents were used following the findings of the paste study.

Visual observations in relation to bleeding, segregation, compatibility, finishability and cohesiveness of concretes were also carried out during placement and compaction of concrete. These are included in Table 6.1 and Table 6.2 and do not raise any concern.

6.2 **Strength characteristics**

The compressive strength development of CSABg recycled aggregate concretes at w/c ratio of 0.55, 0.65 and 0.75 is shown in Figure 6.1. The relative impact of recycled aggregate content on the compressive and flexural strength of concrete at 28 days is shown in Figure 6.2 and Figure 6.3, respectively.

The primary advantage of CSA over PCs is the rapid strength development. As Figure 6.1 shows, all CSABg recycled aggregate concretes reached approximately 75-90% of their 28-day compressive strength at 3 days. At 7 days this was increased to about 90-100%. The use of 50% v/v CDWA in the concrete mix had only a slight detrimental effect on the 28-day compressive cube strength of concrete at all w/c ratios, following a similar trend to that noted in Chapter 5 and in other studies. The strength reduction was in the range of 13-17%, slightly higher from that of CEM I GRA50CDWA which was up to 13%. As compressive and flexural strength are related, the latter which accounts for 6% to 9% of compressive strength also found to decrease with the addition of 50% v/v CDWA. However, the reduction noticed was only in the range of 2-6%, with the 0.75 w/c ratio strength loss at the higher end of this, and can be generally regarded as insignificant.

The relationships between compressive and flexural strength of concrete and w/c ratio are given in Figure 6.4. Similar to PC-based concretes, both strengths have consistently increased with the decrease of w/c ratio from 0.75 to 0.55 and hence it is believed that the reduction of voids as a result of lowering w/c ratio to be mainly responsible for the increase in strength. However, this relationship has not always be consistent within literature (Scherer, 2004a,b, 1999; Dachtar, 2004; Winnefeld and Lothenbach; 2010). The contribution of ettringite to compressive strength may be affected by the interactions between crystal sizes and void content and the amounts of anhydrous particles. The relationship between 28-day compressive and flexural strength was also found to be linear with a correlation factor of about 0.94.

Table 6.1: Fresh properties of CSABg recycled aggregate concretes

Mix Name	w/c ratio	Plastic density, kg/m³	SP[1], % of cement content	Slump, mm		Bleeding	Segregation	Finishability	Compactability	Cohesiveness
				5 min	45 min					
CSABg GRA	0.55	2345	0.34	155	80	Slight	None	Good	Good	Good
CSABg GRA50CDWA		2340	0.34	165	85	Slight	None	Good	Good	Good
CSABg GRA	0.65	2355	0.37	145	85	Slight	None	Good	Good	Good
CSABg GRA50CDWA		2335	0.37	170	90	None	None	Good	Good	Good
CSABg GRA	0.75	2345	0.4	115	70	None	None	Good	Good	Good
CSABg GRA50CDWA		2325	0.4	125	70	None	None	Good	Good	Good

[1] The superplastiser dose was adjusted to achieve S3 slump class

Table 6.2: Fresh properties of CSABg/FA and CSABg/GGBS concretes

Mix Name	w/c ratio	Plastic density, kg/m³	SP[1], % of cement content	Slump, mm		Bleeding	Segregation	Finishability	Compactability	Cohesiveness
				5 min	45 min					
CSABg/15FA GRA	0.45	2365	0.30	170	85	None	None	Good	Good	Good
CSABg/15FA GRA	0.55	2360	0.34	160	80	None	None	Good	Good	Good
CSABg/15GGBS GRA	0.45	2360	0.30	140	65	None	None	Good	Good	Good
CSABg/15GGBS GRA	0.55	2365	0.34	145	70	Slight	None	Good	Good	Good

[1] The superplastiser dose was adjusted to achieve S3 slump class

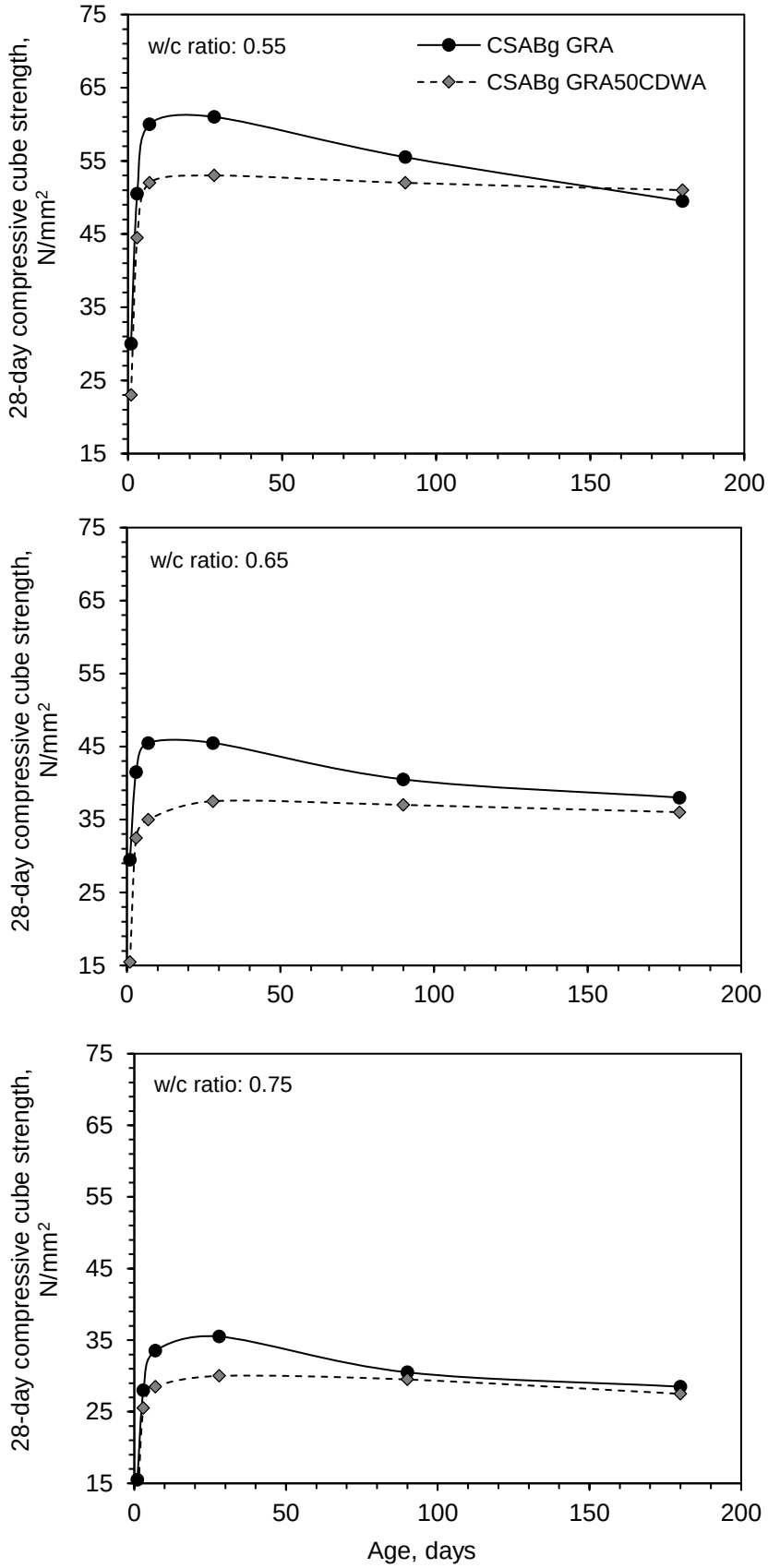


Figure 6.1: Development of compressive cube strength of CSABg recycled aggregate concretes at w/c ratios of 0.55-0.75

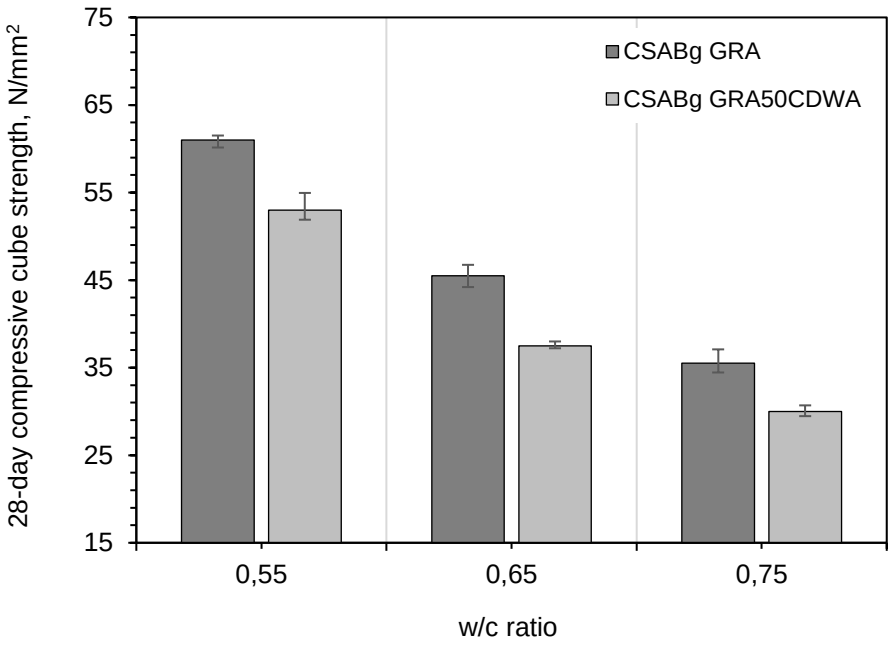


Figure 6.2: Comparison of 28-day compressive cube strength of CSABg recycled aggregate concretes at w/c ratios of 0.55-0.75 (error bars show minimum and maxium values)

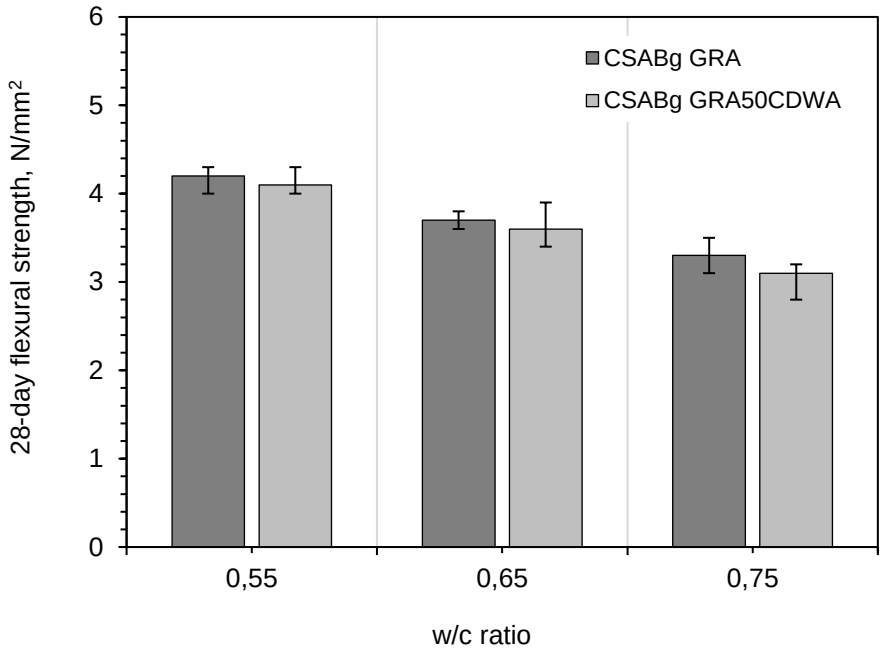


Figure 6.3: Comparison of 28-day flexural strength of CSABg recycled aggregate concretes at w/c ratios of 0.55-0.75 (error bars show minimum and maxium values)

After 28 days no further strength development was observed due to the slow hydration of the belite phase (C_2S content was 20.6%), as suggested by Ioannou et al. (2015) and Quillin (2001; 2007). Instead, all reference CSABg concrete samples experienced a significant loss of strength at all w/c ratios. As Figure 6.5 shows, the strength loss has increased with w/c ratio and curing period. This was in the range of 9-14% and 20-23% for 90 and 180 days, respectively. It is not clear why this occurred, but microcracking induced expansion was most likely responsible for the strength loss. The paste study, however, didn't show any significant expansion strains for gypsum contents up to 2% by mass of CSAB. Another possible cause could be the changes to chemical nature or shape of particular phases. On the contrary, CSABg GRA50CDWA concretes mixes maintained their 28-day strength almost constant with time. The marginal strength loss noticed can be ignored since it falls within the test variability. These concretes were designed with 1.5% w/w gypsum addition compared to 2% w/w used for CSABg GRA concretes to account for soluble sulfates within recycled aggregates. Although the difference was small, it might has reduced the extent of microcracking or phase conversion.

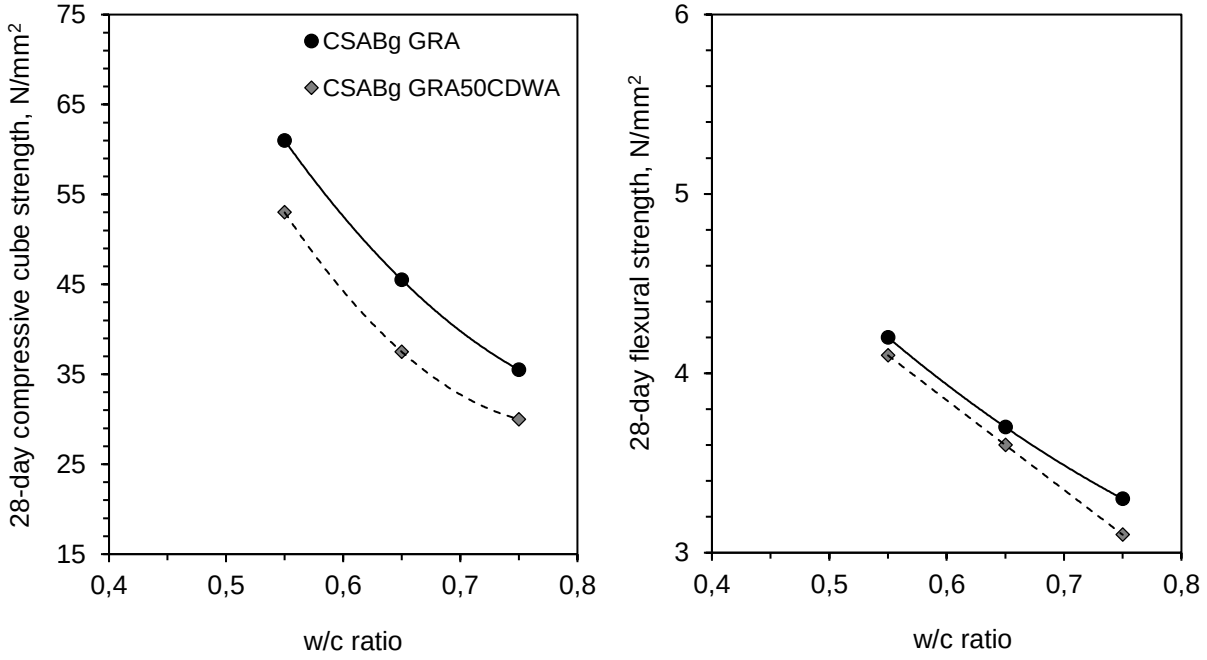


Figure 6.4: Relationship between concrete strength and w/c ratio for the test CSABg recycled aggregate concretes

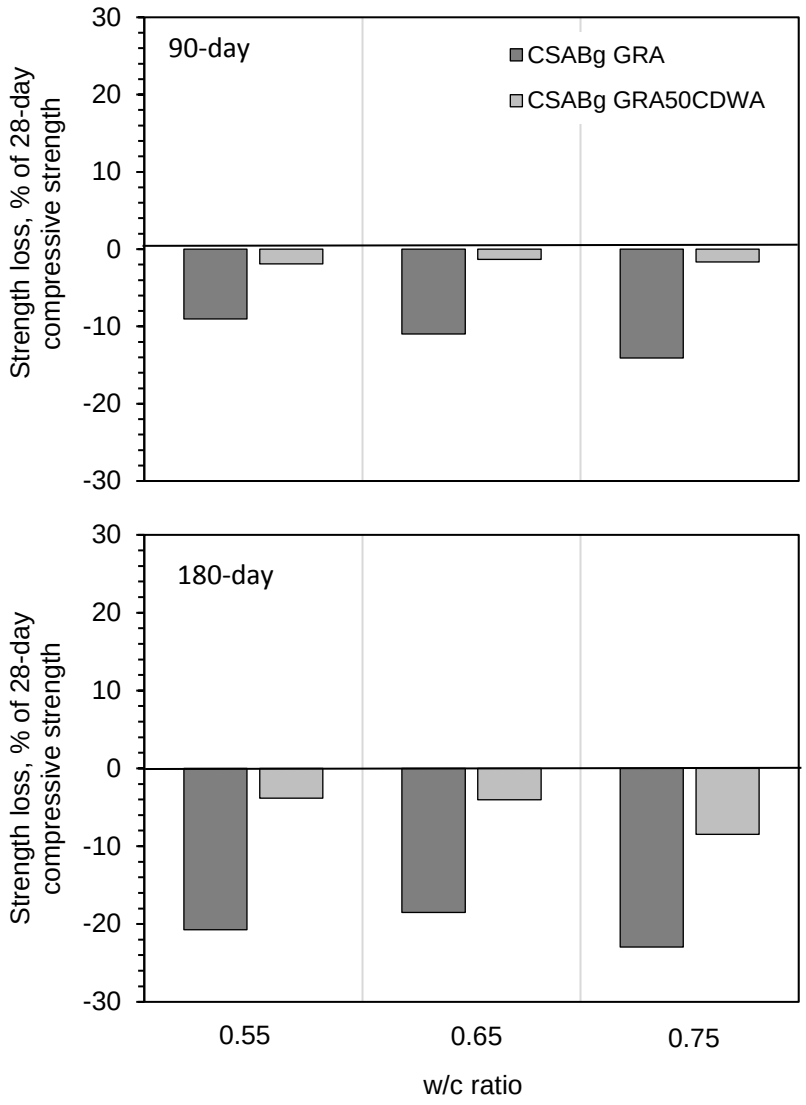


Figure 6.5: Strength reduction at 90 and 180 days of curing as a percentage of 28-day compressive cube strength at w/c ratios of 0.55-0.75

Figure 6.6 shows the compressive strength development of binary cement concretes at w/c ratio of 0.45 and 0.55. Once FA or GGBS were added to the mixes the CSABg/FA and CSABg/GGBS concretes continued to gain strength beyond 28 days or at least to maintain their 28-day strength for a period up to 180 days. In particular, at 0.45 w/c ratio the strength increase noted between 28 and 90 days for CSABg/FA and CSABg/GGBS concretes was 4.5% and 7%, respectively. At 0.55 w/c ratio no further strength development was noted beyond 28 days for both cement combinations. There was a marginal strength loss (≈3%) with the 15% w/w FA addition but can be considered as insignificant. This behaviour is assumed to be due to pozzolanic activity but given the lower availability of lime in CSABg mixes, there is no advantage to increasing the level of pozzolana above 15% as suggested by other (Zivica, 2001; Ioannou, et al., 2014; Guo and Shi, 2016), which contrasts with the PC/pozzolana mixes.

The relationship between strength characteristics of binary cement concretes and w/c ratio is given in Figure 6.7. At an equal w/c ratio, the use of 15% w/w FA or GGBS addition resulted in a reduced 28-day compressive strength. The reduction was approximately 24.5% and 13%, respectively, relatively high considering that the addition level was only 15% w/w.

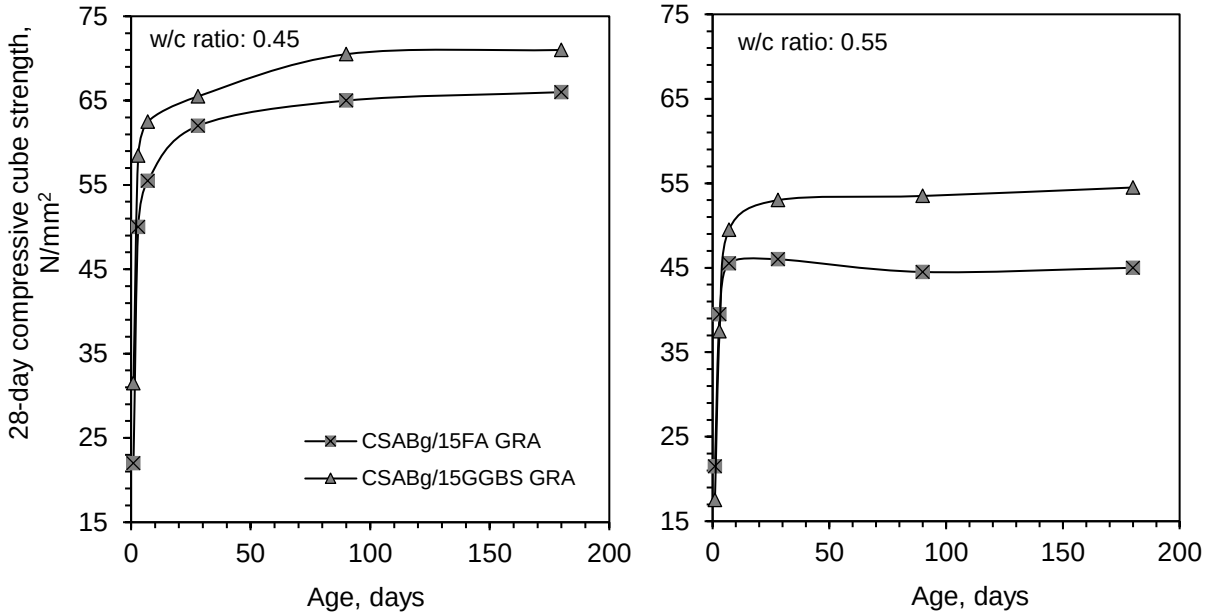


Figure 6.6: Development of compressive cube strength of CSABg/FA and CSABg /GGBS concretes at w/c ratios of 0.45 and 0.55

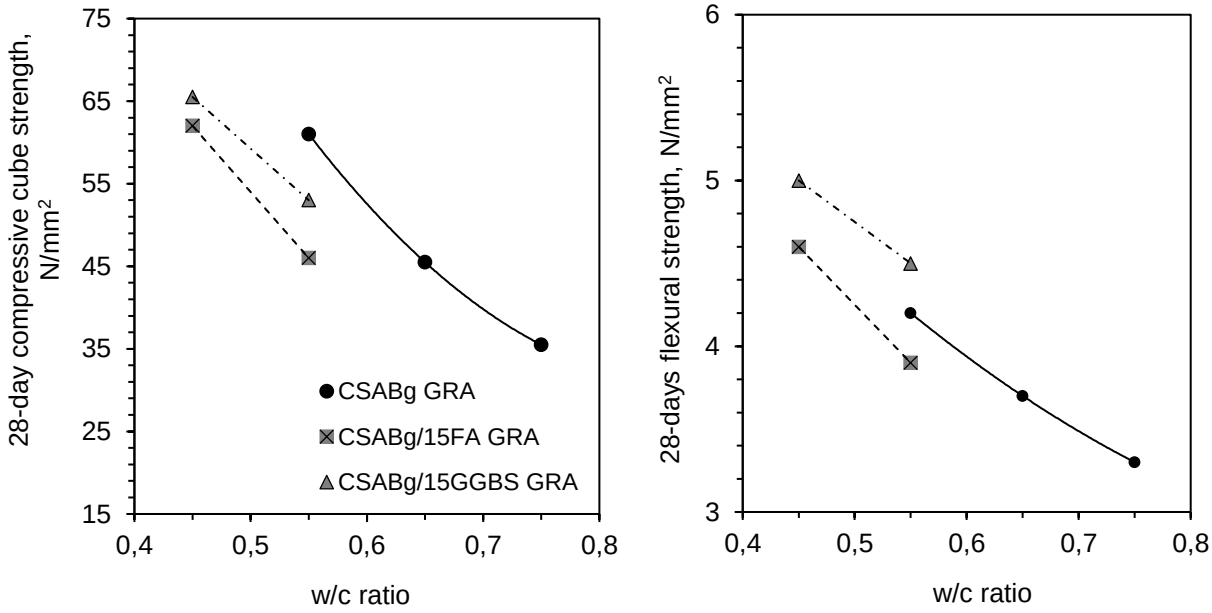


Figure 6.7: Relationship between 28-day strength and w/c ratio for the test CSABg/FA and CSABg /GGBS concretes

6.3 **Drying shrinkage**

Drying shrinkage measurements over a period of 112 days were carried out only for CSABg recycled aggregate concretes and the results alongside with water evaporations are presented in Figure 6.8. The drying shrinkage of all specimens was found to increase with time and stabilise at about 56 days. At an equal w/c ratio, drying shrinkage appeared to increase with w/c ratio and reduce with concrete strength, as Figure 6.9 shows, following similar trends to those noted for CEM I recycled aggregate concretes. Although the CSABg concretes showed a comparable amount of water loss to CEM I concretes, they all exhibit significantly low drying shrinkage values which is probably due to the expansive nature of the CSAB cement achieving shrinkage compensation.

The incorporation of 50% v/v CDWA appeared to have a beneficial effect to shrinkage both at equal w/c ratio and strength. Given that CDWA have met the 0.075% aggregate drying shrinkage limit set in BS EN 12620 (BSI, 2002) and this trend was consistent throughout all w/c ratios, the marginally reduced values may be due to a small alteration of w/c ratio, as a result of a higher CDWA water absorption.

6.4 **Absorption**

The ISAT-10min results for all test concretes in relation to w/c ratio and 28-day strength are given in Figure 6.10. In line with the expected behaviour, all concretes gave a decreasing ISAT-10 value with decreasing w/c ratio and increasing compressive strength. The 50% v/v CDWA addition had a detrimental effect on concrete absorptivity, both at equal w/c ratio and strength, resulting in ISAT-10 values increased by approx. 25%. A similar increase has been recorded for CEM I GRA50CDWA concretes.

For a particular w/c ratio, the addition of FA and GGBS to the CSABg mixes resulted in higher or slightly reduced values, respectively, but for the same strength both ISAT-10 values were reduced. The latter was mainly the effect of reduced w/c ratio used rather than any beneficial effect of FA or GGBS incorporation.

The data of the present study do not reflect the generally expressed literature findings, where CSA-based cements are reported to develop a dense and biomodal (disconnected and isolated pores) microstructure earlier than PCs (Glasser and Zhang, 2001; Garcia-Mate, et al., 2013;

Telesca, et al., 2014) and the inclusion of pozzolonaic materials (up to 15%w/w) in the CSAB blend to further reduces the total porosity (Zivica, 2000; Ioannou, et al., 2014). In addition to that when these results are compared with the ranges given in Technical report No. 31 (The Concrete Society, 2008), the water absorptivity of all concretes is characterised as high.

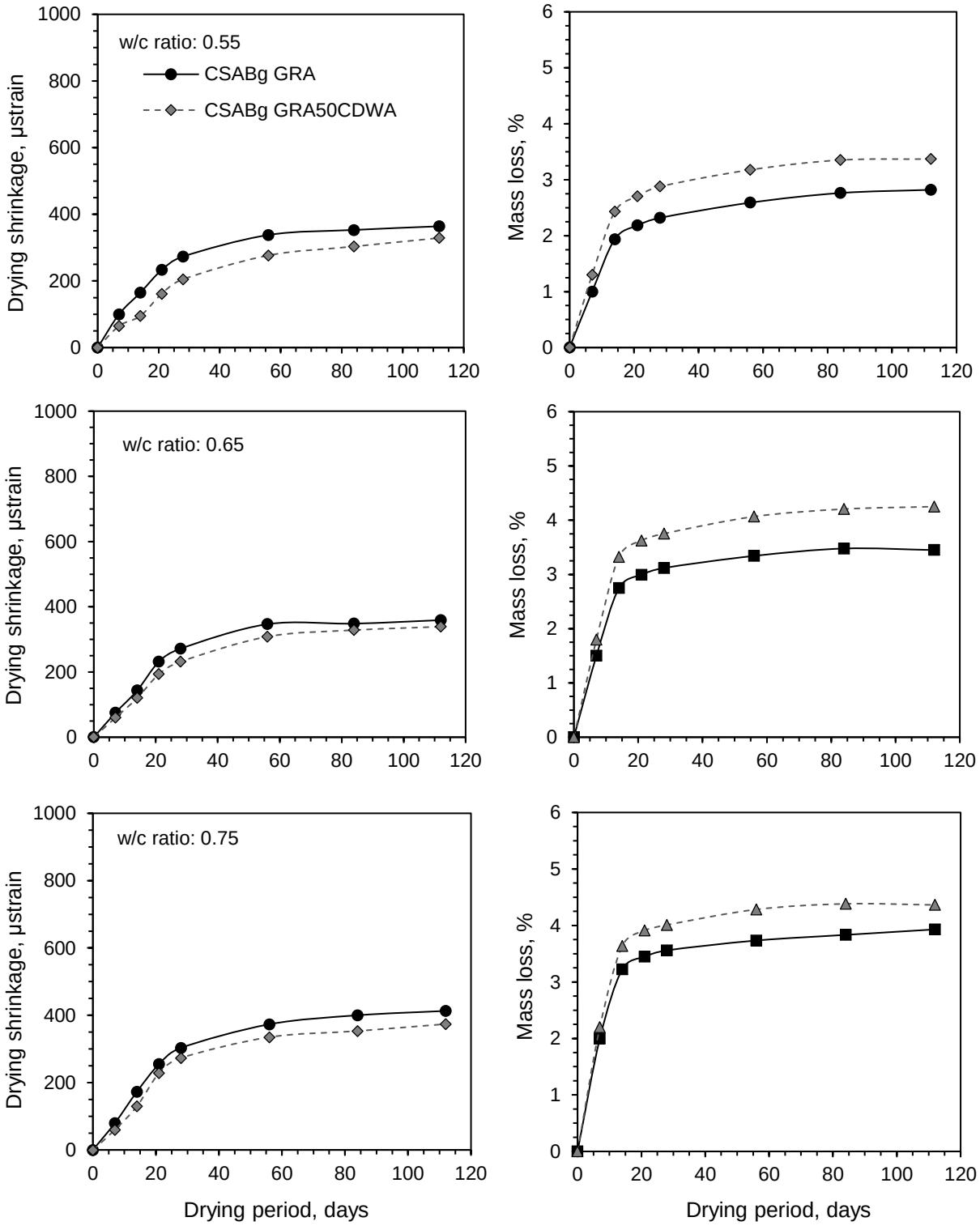


Figure 6.8: Concrete shrinkage development of CSABg recycled aggregate concretes alongside with water evaporation at w/c ratios of 0.55-0. 75.

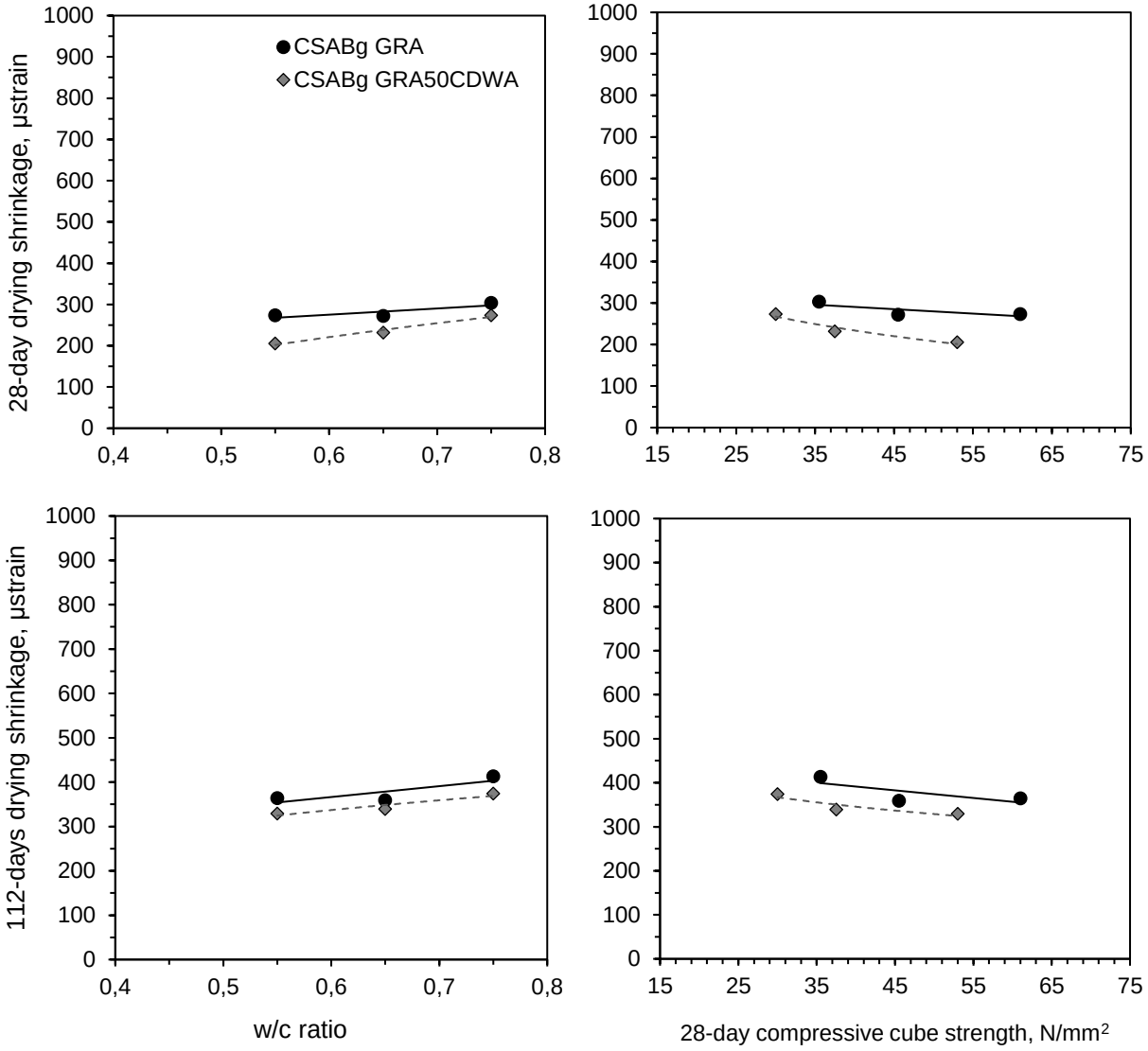


Figure 6.9: Drying shrinkage of CSABg recycled aggregate concretes in relation to (i) w/c ratio and (ii) 28-day compressive cube strength.

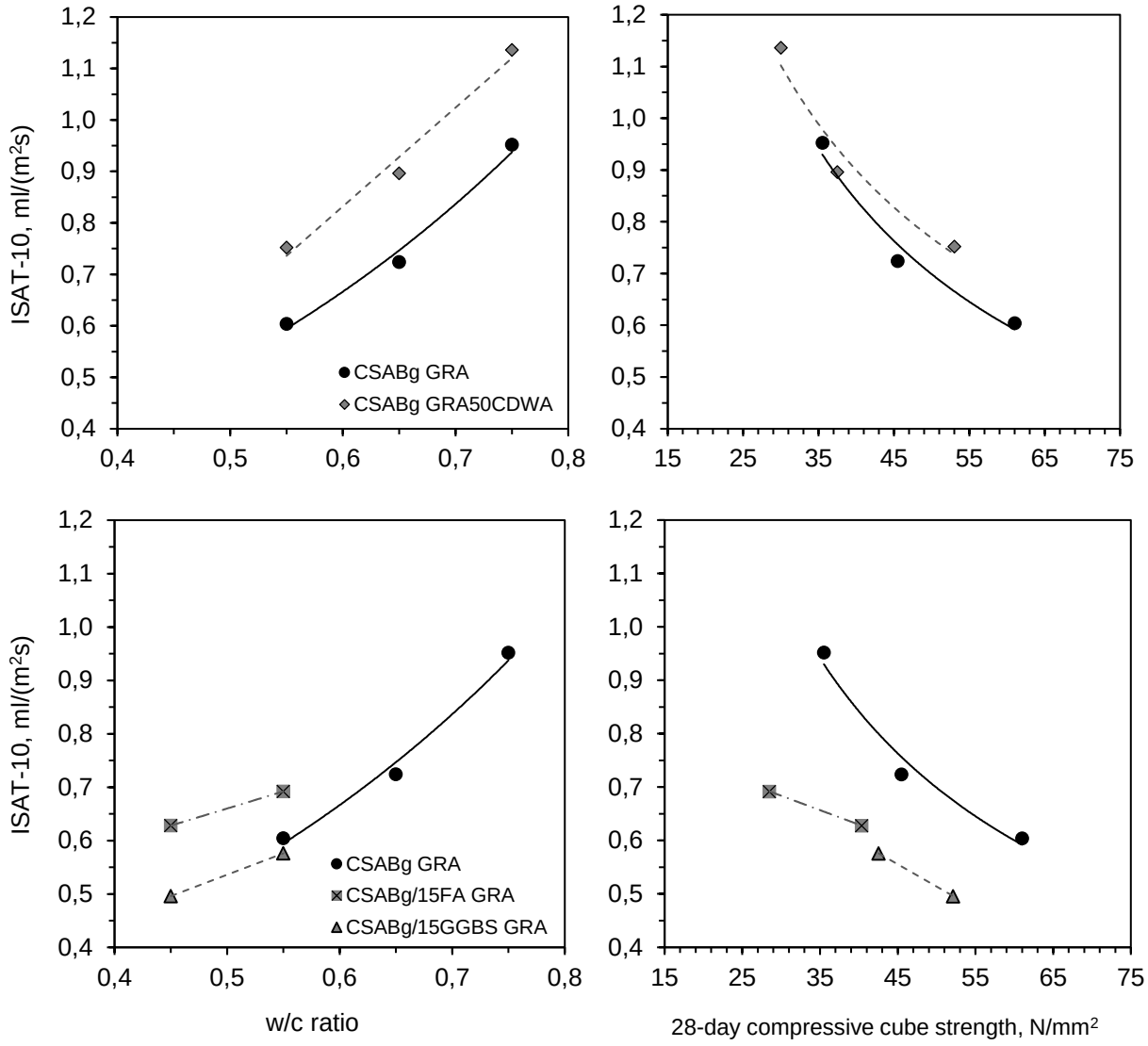


Figure 6.10: Initial surface absorption of CSABg recycled aggregate and binary cement concretes in relation to w/c ratio and 28-day compressive cube strength

Figure 6.11 shows the relationships between initial water absorption and 28-day compressive cube strength for CSABg recycled aggregate and binary cement concretes. As expected, a strong correlation ($R^2 \geq 0.89$) was found between these two properties

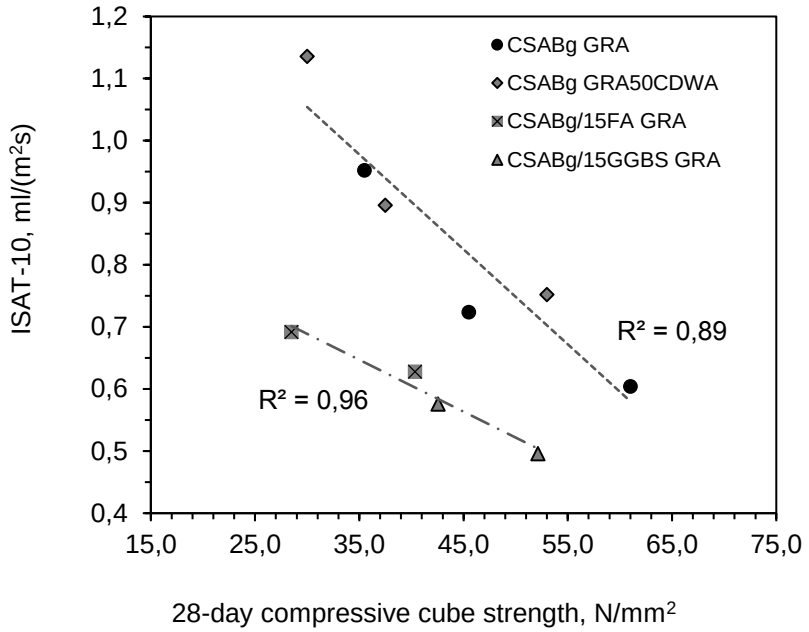


Figure 6.11: Relationship between initial water absorption and 28-day compressive cube strength for CSABg recycled aggregate and binary cement concretes

6.5 **Carbonation**

Carbonation rates of CSABg recycled aggregate concretes at various w/c ratios and measured throughout a period of 20 weeks are shown in Figure 6.12. The relationships between 10-weeks and 20-weeks carbonation depths and (i) w/c ratio and (ii) 28-day compressive cube strength are given in Figure 6.13.

In line with expected behaviour, the carbonation depths of CSABg recycled aggregate concretes increased with w/c ratio and exposure period. At an equal w/c ratio basis, the use of 50% v/v CDWA addition in the mix had no or little detrimental effect to the carbonation resistance of concrete. On an equal strength basis, however, recycled aggregate concrete showed lower carbonation depths. The latter was mainly the effect of reduced w/c ratio required to achieve equivalent 28-day compressive strength.

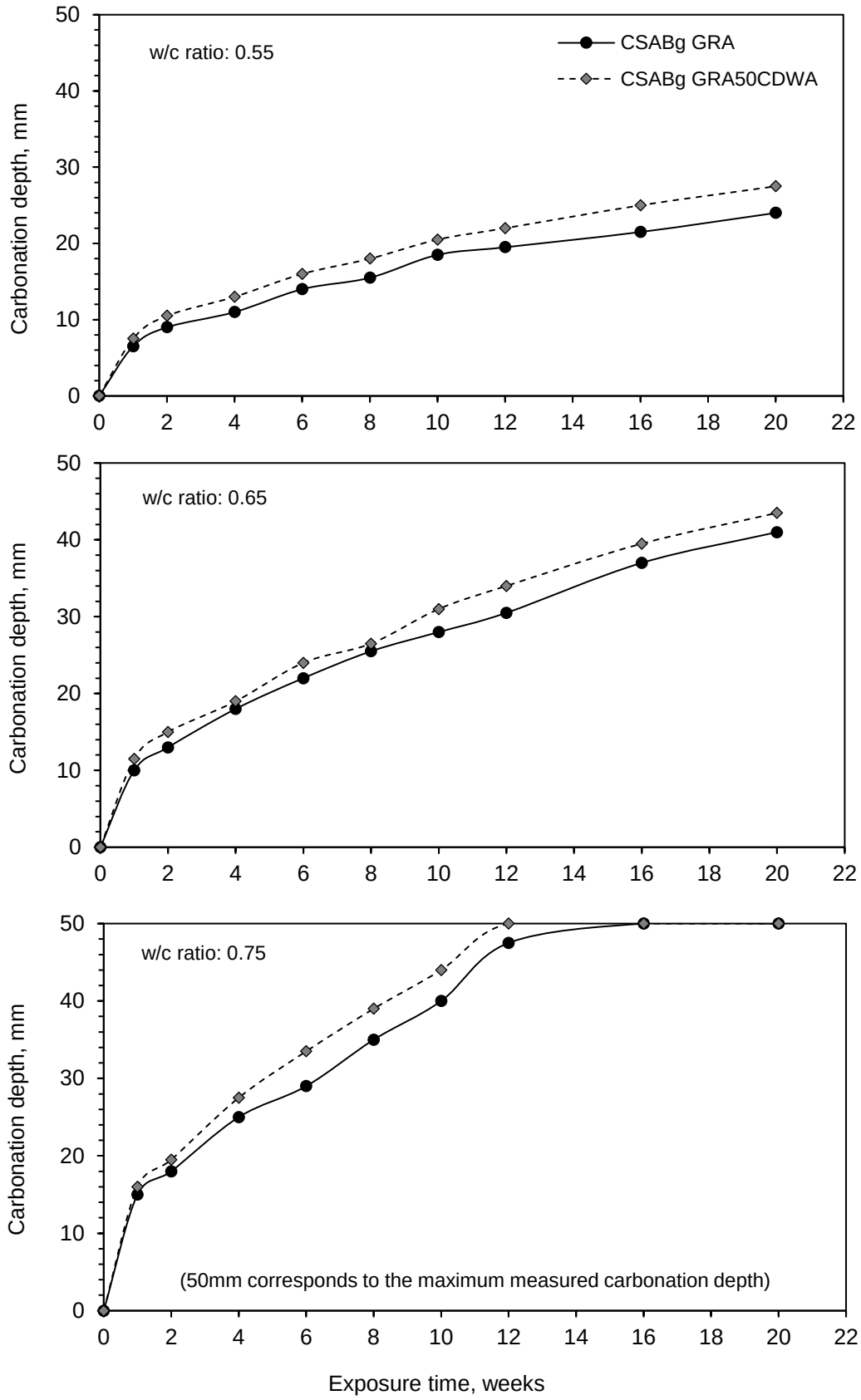


Figure 6.12: Carbonation depths of CSABg recycled aggregate concretes exposed to 4% CO_2 in relation to exposure period for w/c ratios in the range of 0.55 to 0.75

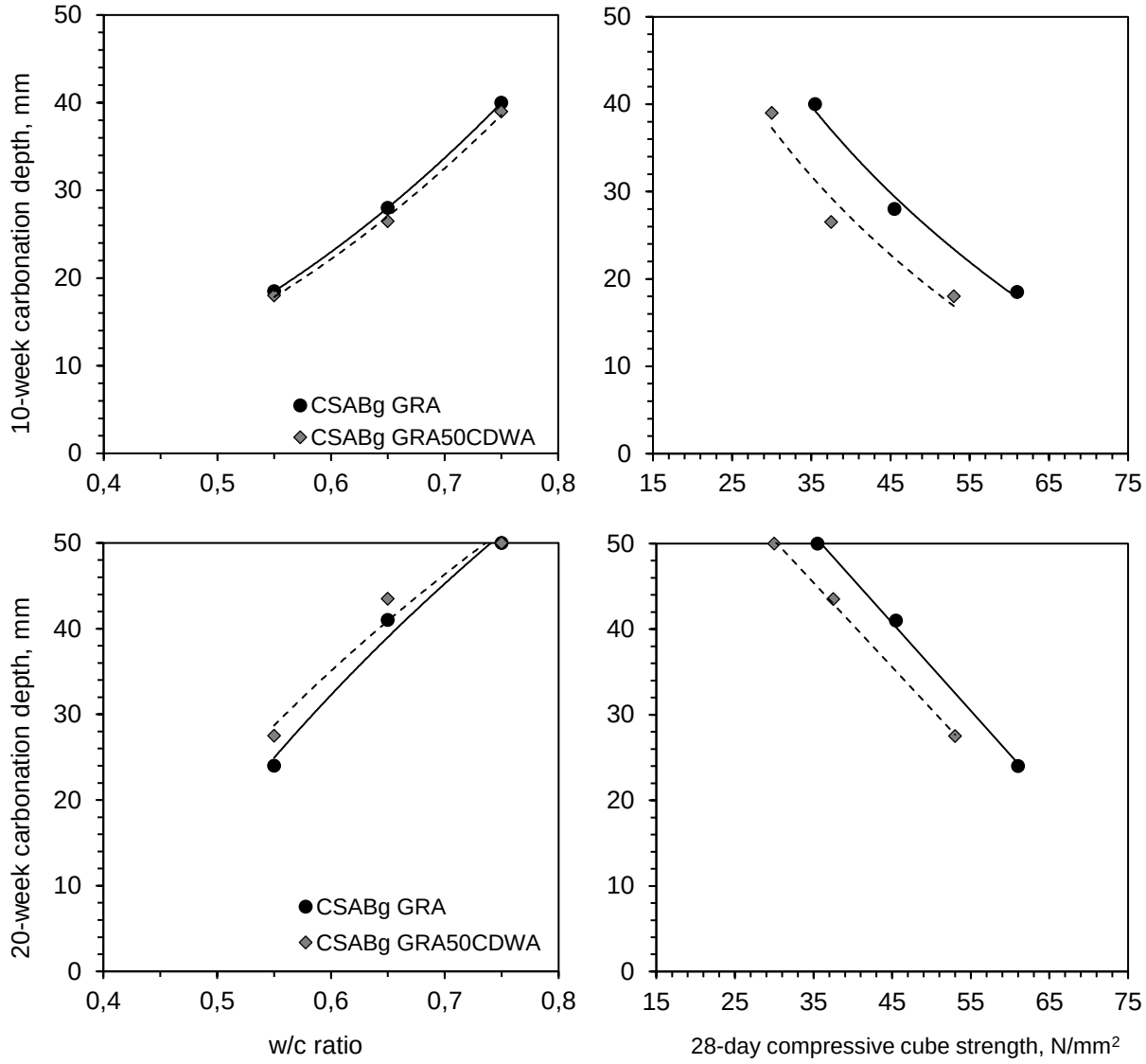


Figure 6.13: Carbonation depths of CSABg recycled aggregate concretes exposed to 4% CO_2 for 10 and 20 weeks in relation to w/c ratio and 28-day compressive cube strength

Figure 6.13 shows similar trends to those noted for CEM I recycled aggregate concretes in Figure 5.17, however, the degree of carbonation was significantly higher. This is mainly due to the low alkalinity of the CSAB cement, which was found to be around pH12. Another factor could be the reported low resistance of ettringite to carbonation (Sato and Takebe, 1992; Sherman, et al., 1995, Bertos, et al., 2004). It is also important to note that the carbonation resistance was established using an accelerated test method and it is possible that in naturally-exposed situations this potential risk is reduced.

Carbonation rates of CSABg/FA and CSABg/GGBS concretes at w/c ratios of 0.45 and 0.55 are given in Figure 6.14. The carbonation resistance of CSABg/15GGBS concrete appeared to be marginally better from that of CSABg/FA concrete at both w/c ratios, however the difference

noted falls within the precision limits of the test and from a practical point of view both concretes can be considered of equivalent performance.

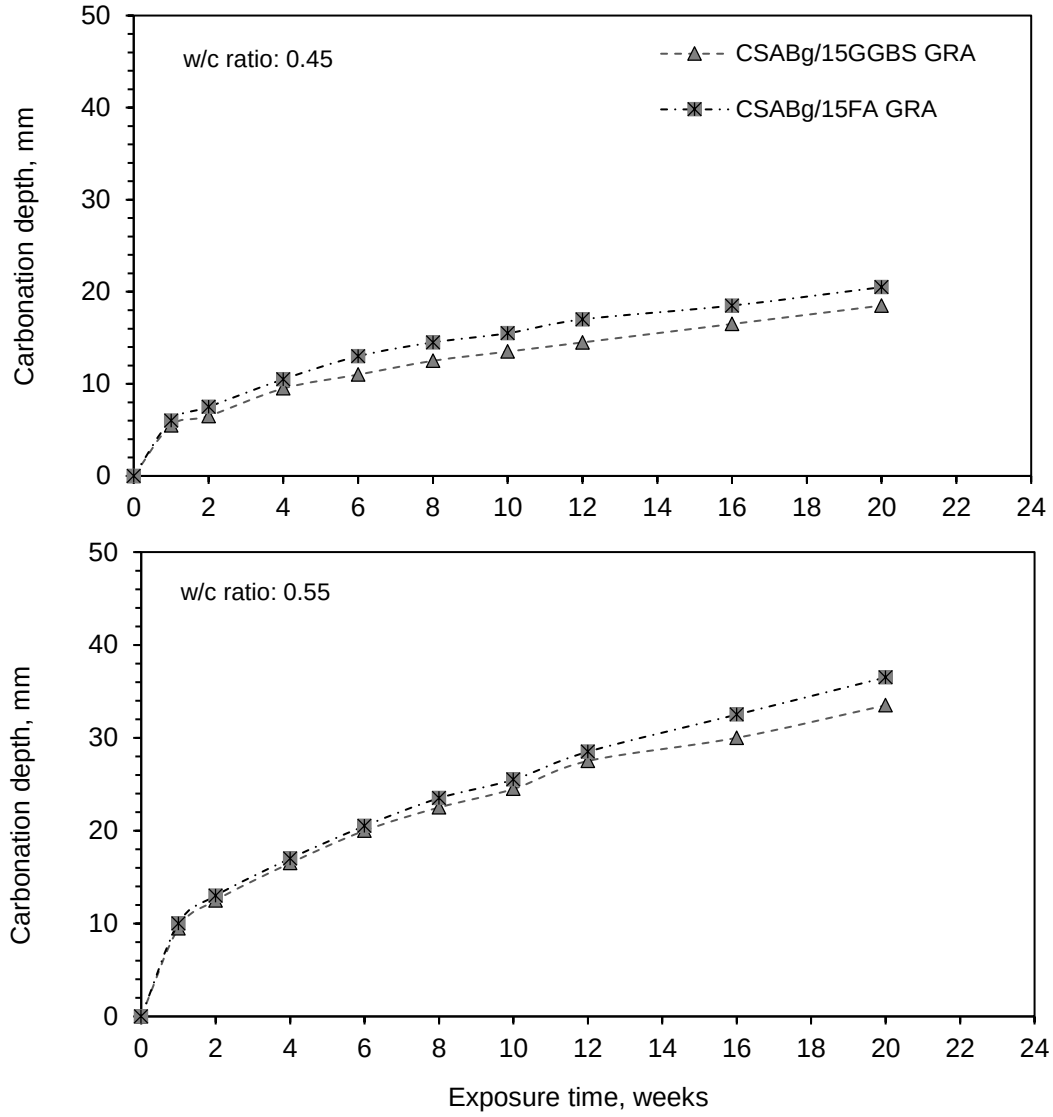


Figure 6.14: Carbonation depths of CSABg/FA and CSABg/GGBS concretes exposed to 4% CO_2 in relation to exposure period for w/c ratios equal to 0.45 and 0.55

Figure 6.15 compares these mixes with the reference concrete on an equal w/c ratio and strength basis. At equal w/c ratio, the carbonation depths of CSABg-based concretes at both exposure periods were also increased with pozzolanic additions, which follows the same trend as for CEM I mixes given lower pore fluid alkalinity of such mixes. For the same strength class, all concretes showed similar carbonation resistance, with the CSBAg/15FA concrete displaying marginally lower values. Clearly the strength results reported above shows that FA and GGBS did contribute to long-term strength development and this would suggest that porosity has been reduced. However, these data suggest that this was insufficient to offset the low alkalinity of

CSA cements, and any improvement noticed was due to the reduced w/c ratio and the higher binder content.

For PC-based concretes the two factors influencing most carbonation are concrete strength and permeability. This appeared to be also the case for CSABg-based concretes, where strong relationships (Figure 6.16) were obtained between the degree of carbonation at both testing ages and 28-day compressive strength or ISAT-10 values ($R^2 \geq 0.73$).

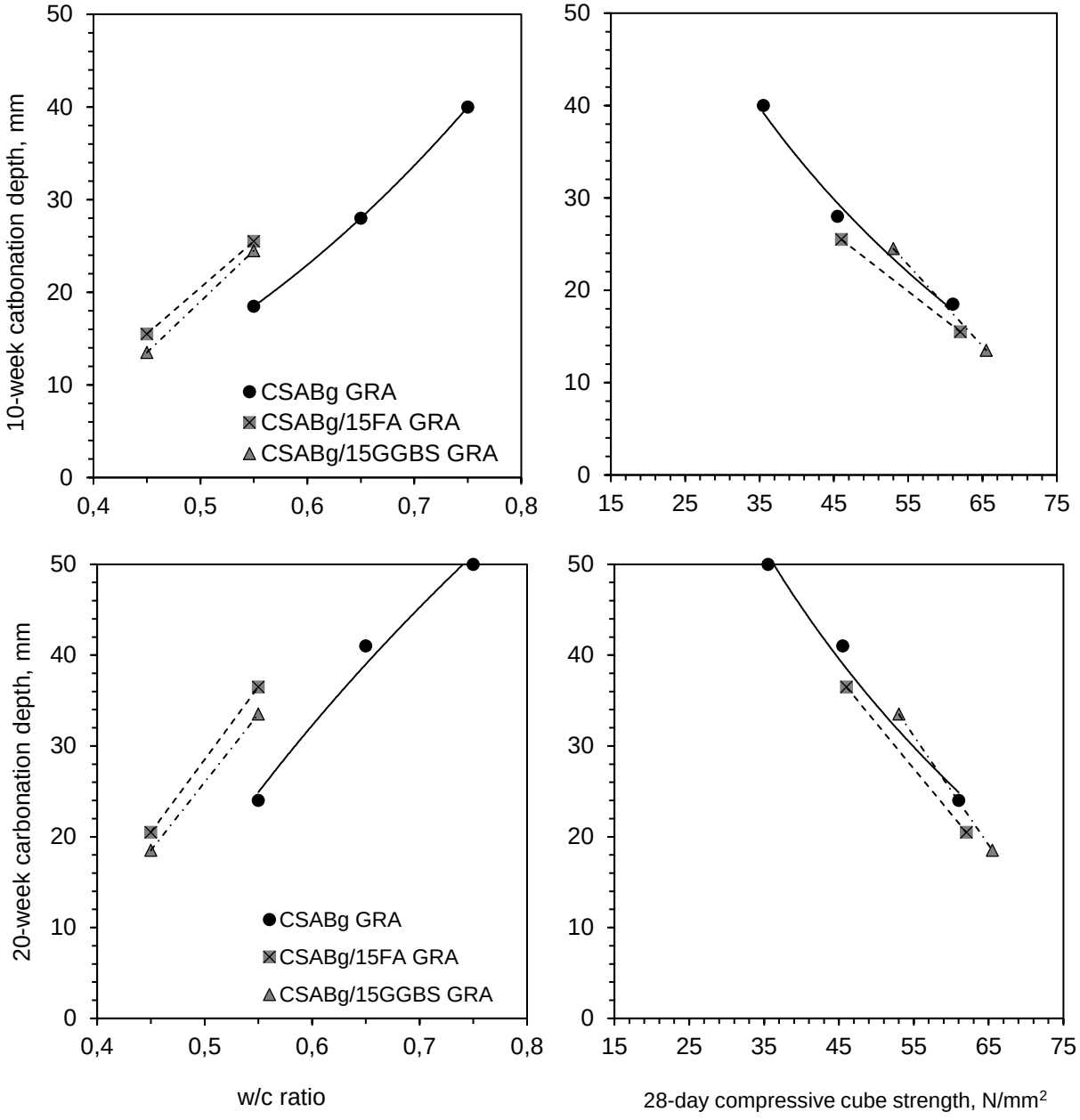


Figure 6.15: Carbonation depths of binary cement concretes exposed to 4% CO_2 for 10 and 20 weeks in relation to w/c ratio and 28-day compressive cube strength

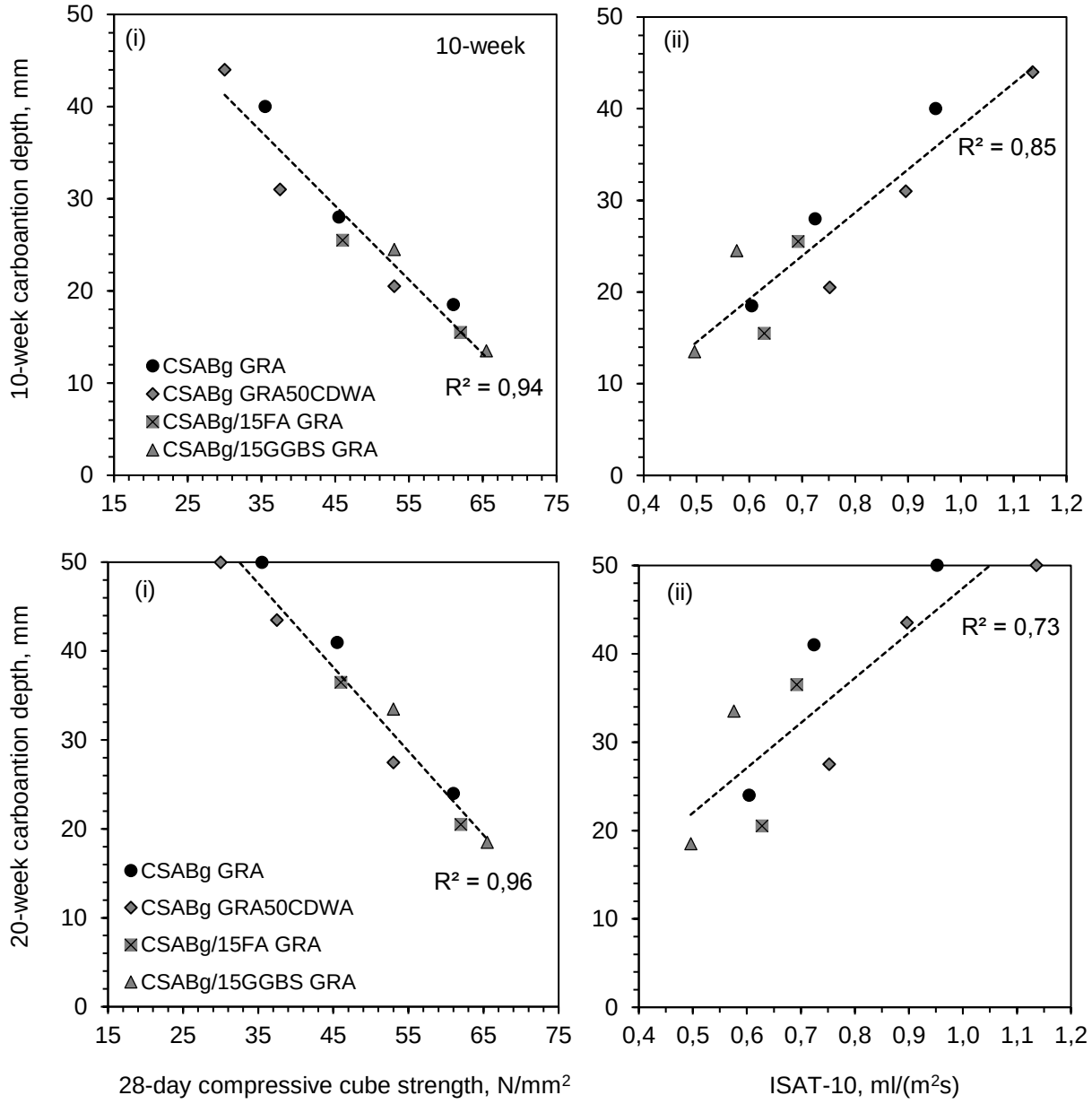


Figure 6.16: Relationship between carbonation depth at 10 and 20 weeks of exposure to 4% CO_2 and (i) 28-day compressive cube strength and (ii) surface absorption (ISAT-10) for all the test recycled aggregate concretes

6.6 **Chloride ingress**

The chloride ingress in CSAB-based concretes was measured using the BS EN 12390 (BSI, 2015) test method, following the findings of the paste study (Section 4.2.6). The non-steady state diffusion coefficients (D_{nss}) of CSABg recycled aggregate and binary cement concretes exposed to 3% NaCl solution for 90 days are given in relation to w/c ratio and 28-day compressive strength in Figure 6.17 and Figure 6.18, respectively. In line with established results these data show a reduction in D_{nss} of all concretes for decreasing w/c ratios and increasing cube strengths.

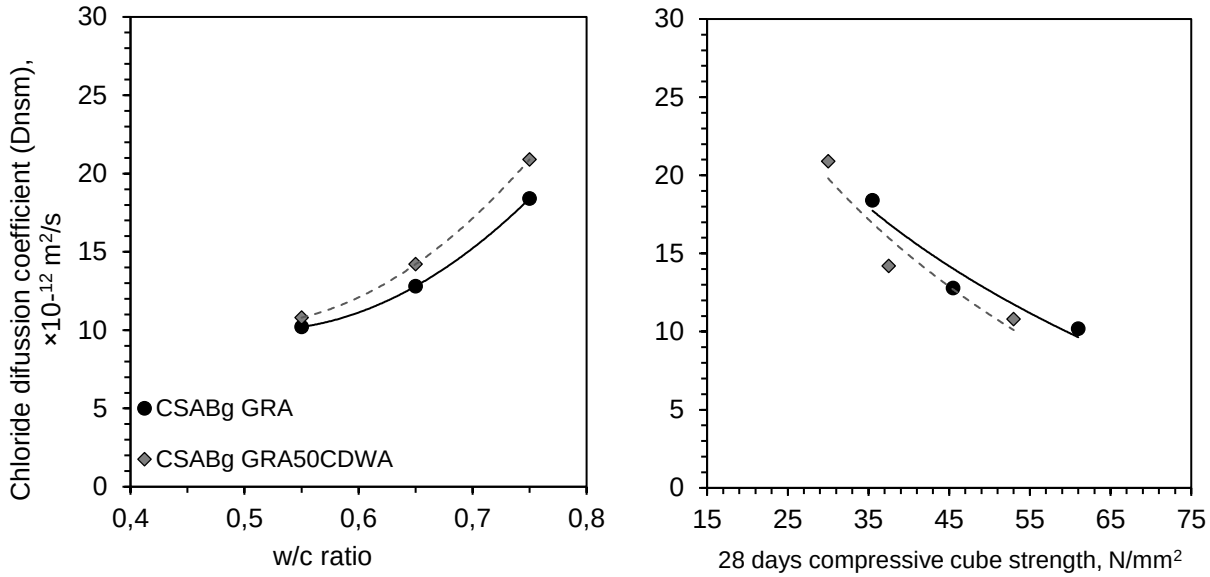


Figure 6.17: Non-steady-state chloride diffusion coefficient of CSABg recycled aggregate concretes in relations to w/c ratio and 28-day compressive strength

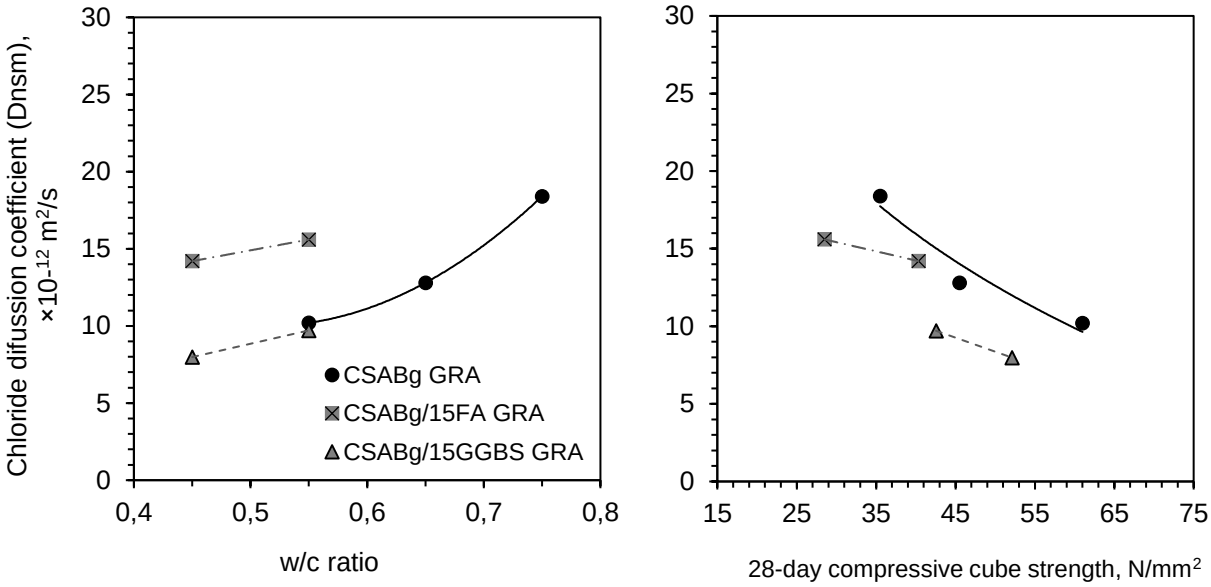


Figure 6.18: Non-steady-state chloride diffusion coefficient of CSABg/FA and CSABg/GGBS concretes in relations to w/c ratio and 28-day compressive strength

In this study a 2% w/w gypsum addition used compared to 10-35% w/w calcium sulfate additions typically found in literature, to promote the formation of AFm phase and enhance the binding ability of the CSAB concretes. Indeed, Figure 6.17 shows a superior resistance of CSABg concretes against chloride ingress and suggest that there is no additional risk of chloride-induced corrosion as shown for carbonation.

At a given w/c ratio Dnss of CSABg GRA50CDWA concrete was only 2.5 ×10^{-12} m^2/s higher than that obtained for CSABg GRA concrete, suggesting not very dissimilar performances. The slightly higher chloride ingress measured can be explained by the increase in porosity and interconnectivity of pore network. At an equal strength basis, however, the reduction in w/c ratio has offset the increase in concrete permeability due to recycled aggregate incorporation in the mix and the resultant concretes displayed a marginally better resistance to chloride ingress.

The results also show that the 15% w/w GGBS addition in the CSABg-base concretes has slightly improved the chloride resistance of concrete at equal w/c ratio, whereas the 15% w/w FA addition resulted in Dnss values increased by 65%. This was largely due to the higher absorptivity of the CSABg/15FA mixes. When the diffusion coefficients are compared for equal strength, both binary cement combinations showed lower Dnss values from reference concrete. This was the combined effect of reduced permeability and increased content of monosulfate (AFm phase), as a result of lower w/c ratio and higher binder content, respectively.

As mentioned previously, chloride ingress depends on the interconnectivity of the pore network in concrete and the binding ability of the hydrated phases. Given that Dnss account for chloride binding effects, there was a correlation between the ISAT-10 values and chloride ingress as shown in Figure 6.19ii. A good correlation was also found between chloride diffusion coefficients and 28-day compressive strength (Figure 6.19i).

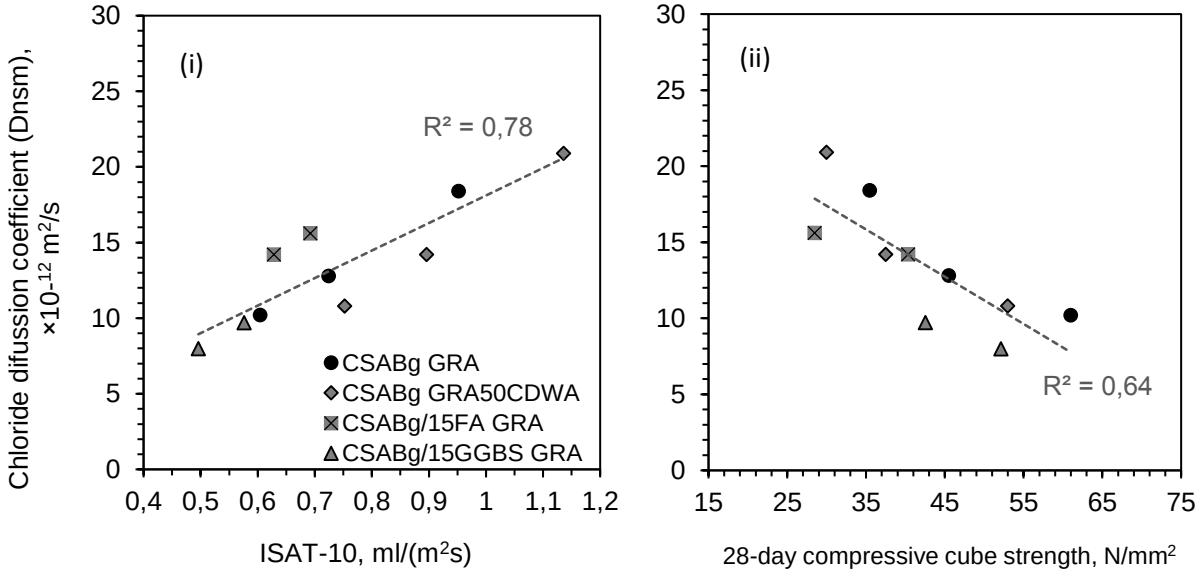


Figure 6.19. Relationship between non-steady-state chloride diffusion coefficient of CSABg/FA and CSABg/GGBS concretes and (i) ISAT-10 and (ii) 28-day compressive cube strength

6.7 **Sulfate resistance**

Sulfate expansion (as percentage of initial length) of CSABg recycled aggregate concretes in relation to exposure period and testing conditions (fully immersed and cycling wetting and drying) are shown in Figure 6.20 for the three w/c ratios considered. The relationships between 60-week expansions and w/c ratio are given in Figure 6.21. Those with 28-day compressive cube strength are shown in Figure 6.22.

The general held view of CSA cement systems is that provided that the ettringite phase is fully formed during hydration there is no material able to react with external sulfate sources and form expansive products at later ages (Ioannou, et al., 2015; Bescher, et al., 2016). In these study, however, the mixes were deliberately 'under-sulfated' and, as shown in Figure 6.20, sulfate expansion of the test concretes did occur with time. The results indicate a general trend of increasing expansion with exposure time and reducing w/c ratio. Normally as the mix w/c ratio is reduced the ingress of sulfates ions is slowed by reducing porosity and permeability but where such mixes have a higher cementitious content (rather than a reduced water content) this can result in the formation of more phases prone to become expansive, albeit within a denser matrix. This seems to be the case for these concretes.

Regarding the two different testing scenarios, the test concretes performed similarly. Only when the 0.75 w/c ratio concrete mixes exposed to a wetting and drying sulfate environment the expansion values were slightly increased. This suggests that the higher porosities of these mixes enabled the sulfates salts to penetrate to a greater depth and its crystallisation to cause further expansion in addition to the chemical one.

For all w/c ratios, CSABg GRA50CDWA concrete exhibited lower expansion from the reference CSABg GRA concrete throughout the 60-week test period and for both testing conditions. It is not clear why this occurred, but possible causes could be w/c ratio changes due an underestimation of CDWA water absorption. Based on the strength data shown above, it is also likely that microstructural changes within the paste matrix of CSABg GRA concrete have caused microcracks and facilitated in that way the ingress of sulfates. In any case, however, the differences noticed between these concretes are minor, given that the total expansion is less than 0.02%, and found to reduce with increasing the w/c ratio. At an equal strength basis, CSABg GRA50CDWA concretes displayed similar or slightly lower expansions compared to reference CSABg GRA concrete at both testing scenarios.

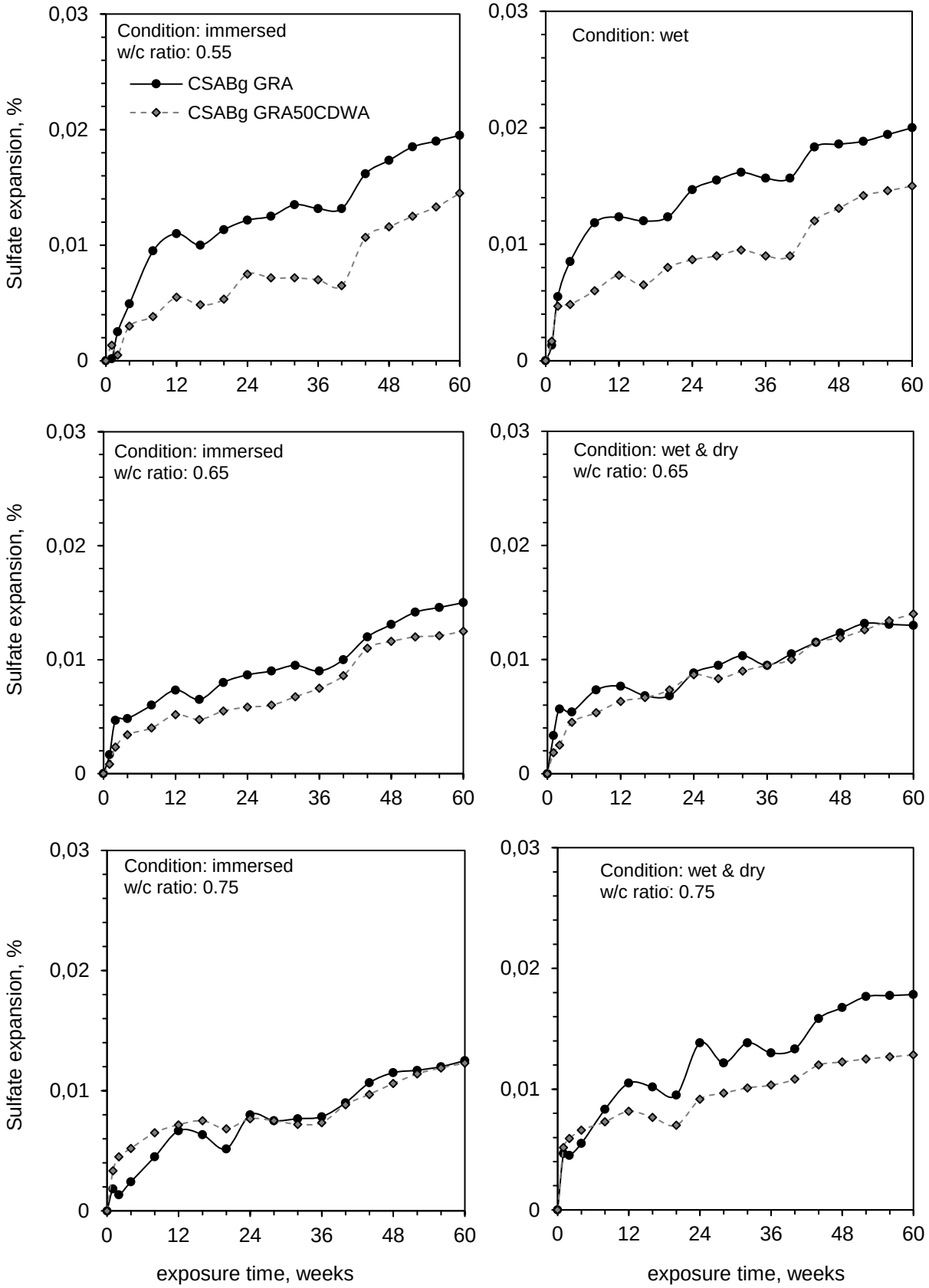


Figure 6.20: Relationship between sulfate expansion of CSABg recycled aggregate concretes and exposure time for w/c ratios equal to 0.55, 0.65 and 0.75

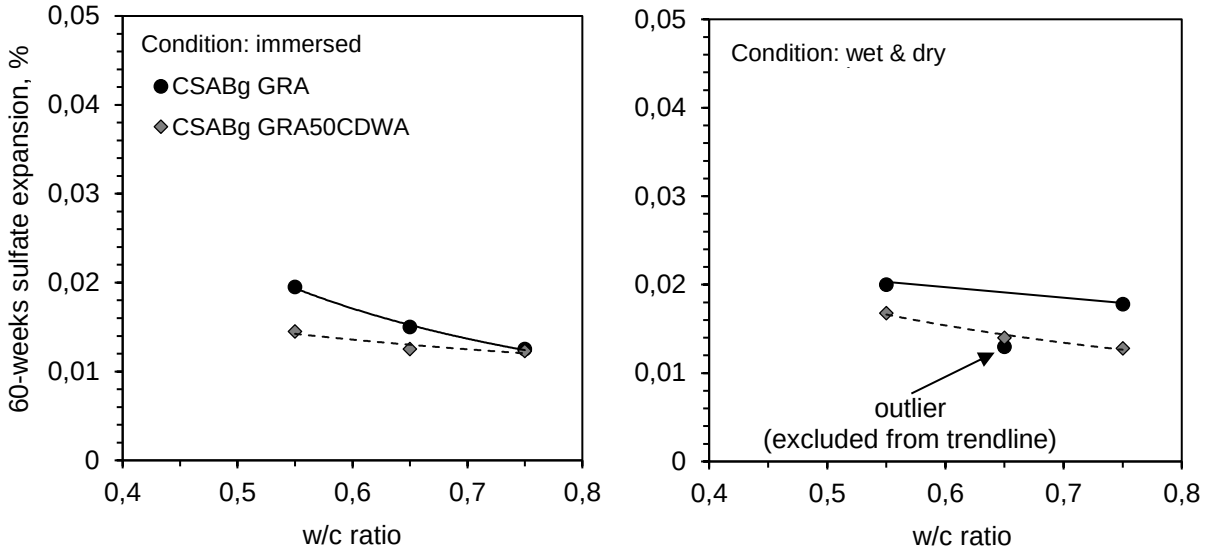


Figure 6.21: Relationship between 60-weeks sulfate expansion of CSABg recycled aggregate concretes and w/c ratio of concretes for the two testing conditions

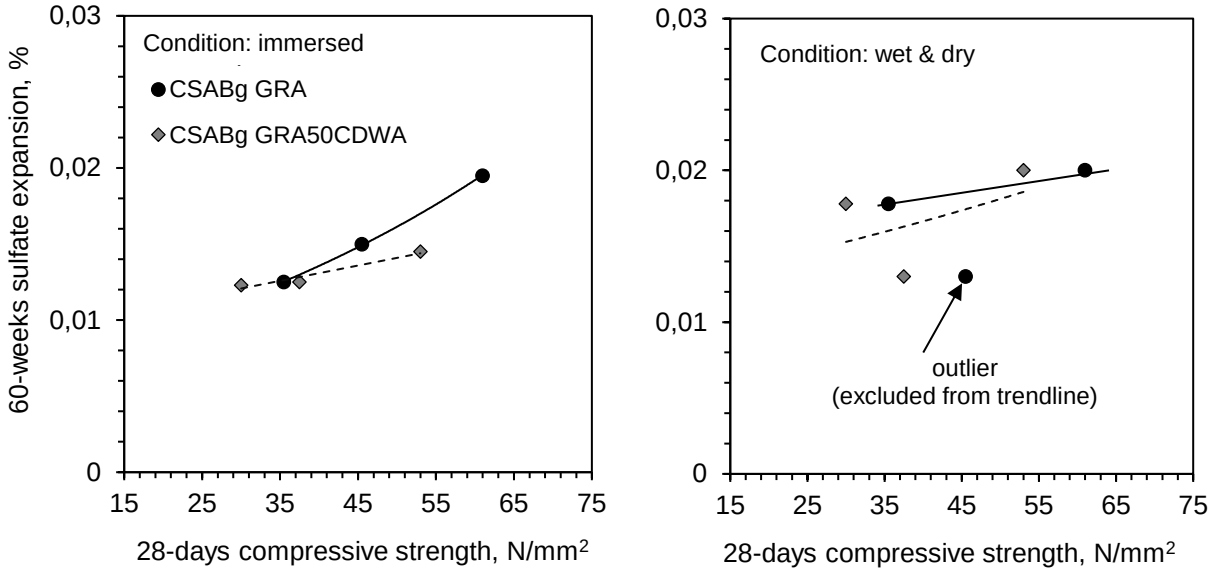


Figure 6.22: Relationship between 60-week sulfate expansions of CSABg recycled aggregate concretes and 28-day compressive cube strength of concretes for the two testing conditions

Typically, pozzolanic materials are used with cement combinations with PC to provide a degree of protection against sulfate expansion by both diluting the amount of CAH and reducing the porosity of the concrete. This appears to be also the case in CSAB-based systems. Figure 6.23 shows the sulfate expansion (as percentage of initial length) of CSABg/15FA and CSABg/15GGBS concretes in relation to exposure period and testing conditions for the two w/c ratios considered. Both pozzolanic additions resulted in equivalent performance, with the GGBS addition resulting only in marginally lower expansion values.

Figure 6.24 shows the relationship between 60-week expansion and w/c ratio for the two testing scenarios. For the same w/c ratio (i.e. 0.55) binary cement concretes displayed marginally lower expansions at both conditions. This behaviour is associated with the dilution of the CSAB content. Indeed, CSABg/15FA and CSABg concretes at w/c ratios of 0.55 and 0.65, respectively, comprise equal CSABg contents and exhibit comparable expansions. A similar relationship could be also seen between CSABg/15GGBS and CSABg at w/c ratios of 0.45 and 0.55, respectively. For a specific strength (Figure 6.25) CSABg concrete samples been fully immersed to sulfate solution exhibited similar or lower expansions followed by CSABg/15GGBS and CSABg/15FA concretes. In cyclic wetting and drying exposure conditions, CSABg concrete exhibited the lowest expansion towards higher strengths, but for medium and low strengths CSABg/15GGBS appeared to perform better followed by CSABg and CSABg/15FA concretes.

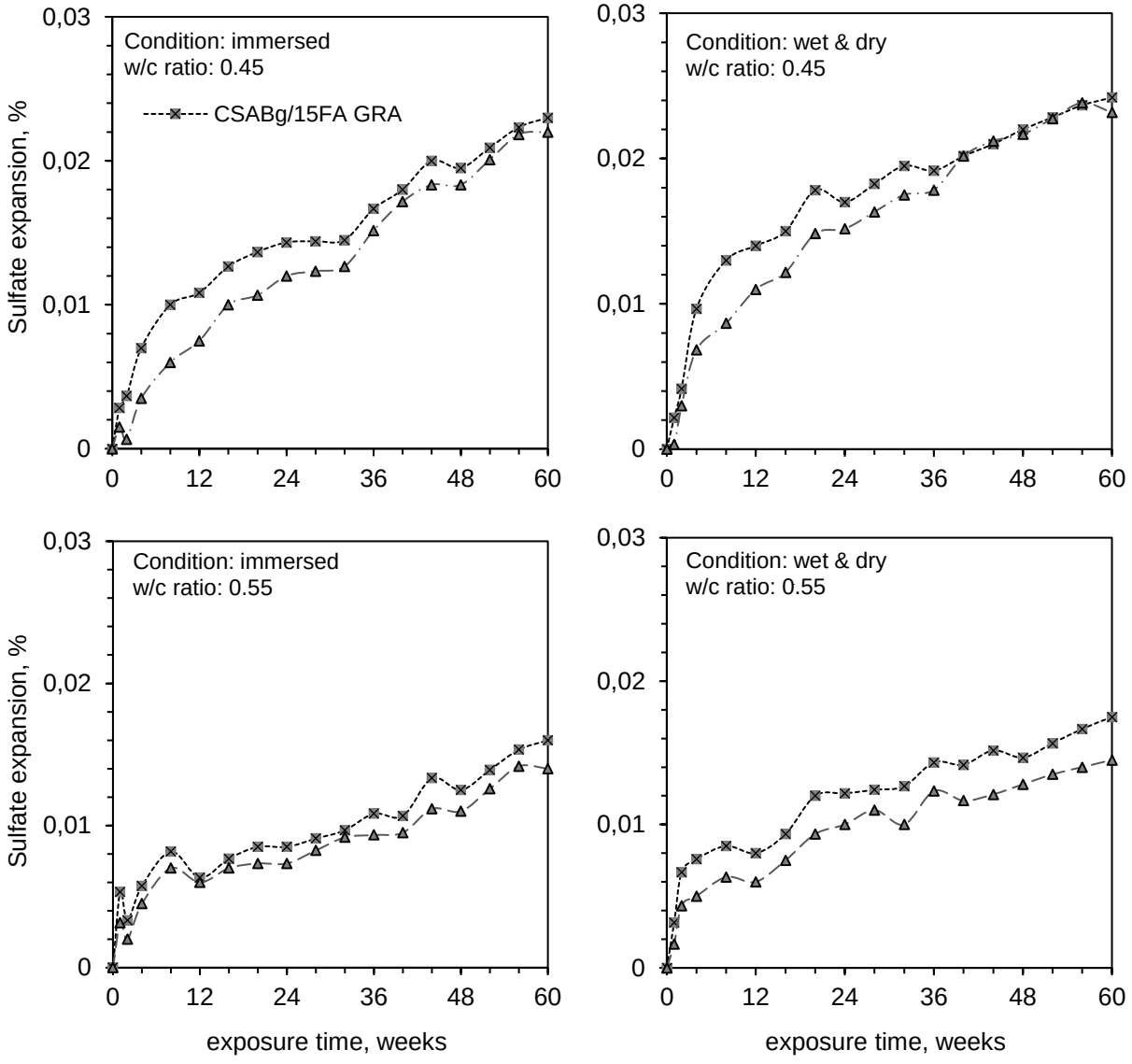


Figure 6.23: Relationship between sulfate expansion of binary cement concretes and exposure time for w/c ratios equal to 0.45 and 0.55

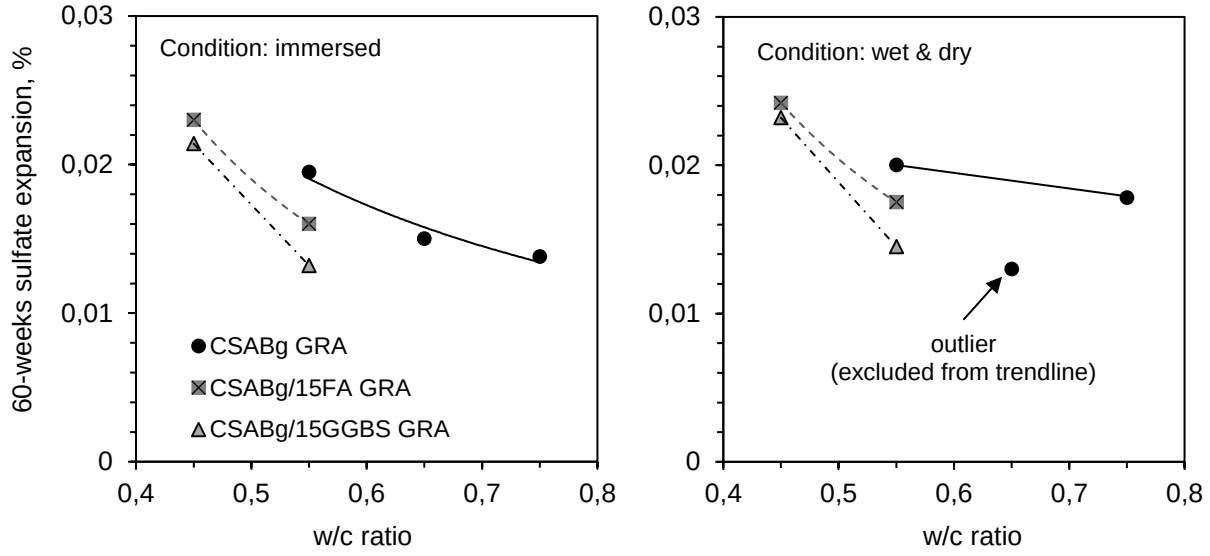


Figure 6.24: Relationship between 60-week sulfate expansion of binary cement concretes and w/c ratio of concretes for the two testing conditions

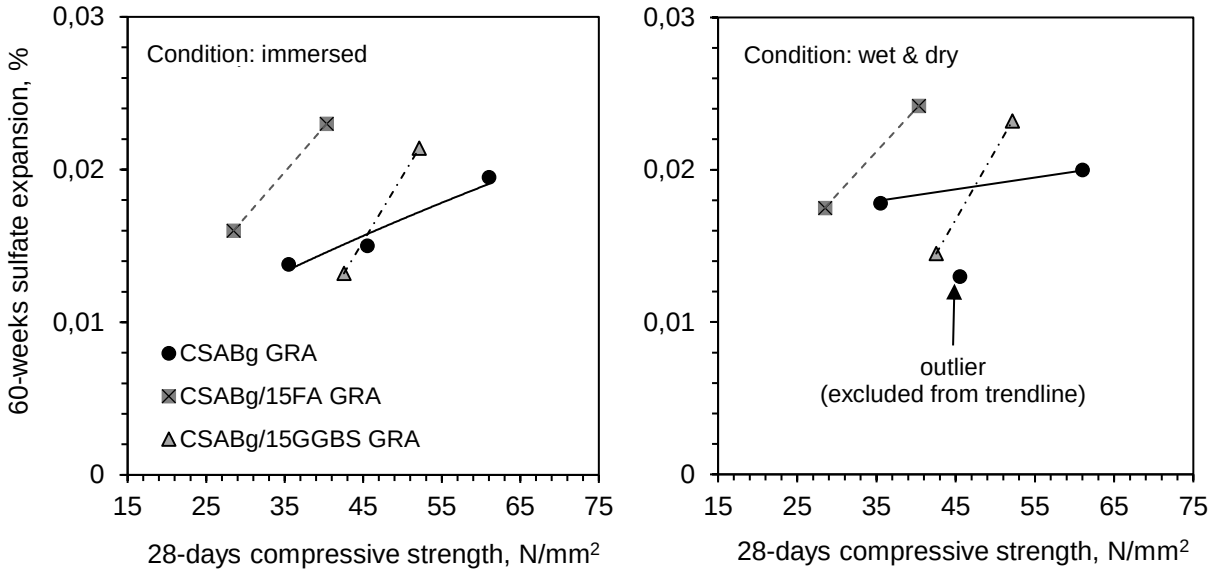


Figure 6.25. Relationship between 60-week sulfate expansion of binary cement concretes and 28-day compressive cube strength of concretes for the two testing conditions

Overall, none of the CSAB-based samples in the current study approached the 0.1% expansion limit suggested by Samarai (1976) and no surface degradation or concrete spalling observed, except some minor discoloration in a few samples from the formation of gypsum (Figure 6.26).

Figure 6.26: Visual appearance of CSAB-based concretes after 60 weeks of exposure to sulfates

6.8 **Conclusions**

6.8.1 Fresh properties

CSAB cement can be used successfully with a compatible polycarboxylate based superplasticiser to produce workable concrete suitable for structural applications. The consistence of concrete was found to improve with the addition of 50% v/v CDWA due to their more spherical shape, whereas additions of 15% w/w FA or GGBS had no or little detrimental effect. In all cases, however, the concretes lost initial workability rapidly. Although there was sufficient time prior to setting to allow laboratory test samples to be compacted and finished, for on-site applications a retarder would be required.

6.8.2 Engineering properties

The early strength development of all CSABg-based concretes was rapid and concretes achieved approximately 75-90% of their 28-day compressive strength within 3 days. However, between 28 and 180 days of standard water-curing CSABg concretes lost up to 23% of their 28-day strength. Although it was not clear what the cause of this was, it is likely that internal cracking due to volume instability was a primary issue. For a given w/c ratio, the addition of recycled aggregates (50% v/v CDWA) or pozzolanic materials (15% w/w FA or GGBS) has resulted in lower strengths but counteracted the strength loss and thus CSABg recycled aggregate and binary cement concretes maintained their 28-day strength almost constant with time. In the former case this was probably because of the lower gypsum addition used to account for soluble sulfates within recycled aggregates, whereas in the latter one it is assumed to be due to pozzolanic activity.

The expansive nature of CSA cements has a shrinkage compensation effect and therefore all drying shrinkage measurements were found significantly lower than those recorded for CEM I concretes, even though the water losses were comparable. The incorporation of 50% v/v CDWA also had a minor beneficial effect in relation to shrinkage both at equal w/c ratio and strength basis. This was attributed to a reduced w/c ratio as the water absorption of CDWA it might has been slightly underestimated.

The generally held view that CSA-based cements develops a dense and biomodal microstructure earlier than Portland cements was not verified in this study. The ISAT-10 values were further increased with the use of CDWA at both equal w/c ratio and strength. This was

expected due to their higher porosities. For a given w/c ratio, the addition of FA or GGBS did not reduce the ISAT-10 values, but for a given strength, the concrete absorptivity of CSABg/15FA or CSABg/15GGBS concretes was improved mainly due to the lower w/c ratio required to give equal strength.

6.8.3 Durability properties

All CSAB-based concretes exhibited a reduced resistance to carbonation. The high rate of carbonation observed is related to the reduced amounts of alkali hydroxides present, as shown in the paste study. For the same w/c ratio, the use of recycled aggregates had little or no detrimental effect to carbonation degree. The effect was more pronounced when FA and GGBS additions were used, given both the higher porosities and the lower alkalinity of those concretes. At equivalent strength, CSABg recycled aggregate and binary cement concretes showed similar or improved performance with the reference CSABg-only concrete mainly because a reduced w/c ratio required to compensate strength loss.

The AFm/AFt ratio of CSABg cement was manipulated in the paste study to favour the former and enhance chloride resistance. Indeed, the data suggests an increased potential for binding chlorides which offsets the higher absorptivity of CSABg concretes and results in a superior performance. At equal w/c ratio, the GGBS addition has slightly improved the chloride resistance of concrete, whereas the FA addition resulted in higher diffusion coefficient values. The latter was largely due to the higher absorptivity of the CSABg/15FA concretes. When the diffusion coefficients are compared for equal strength, both binary cement combinations displayed lower coefficients from reference concrete as a result of the combined effect of reduced permeability and increased content of monosulfate (AFm phase).

Given the AFm content and higher absorptivity of these mixes a decreased resistance to sulfate attack was expected. This seems not to be the case for CSAB concretes, where a superior resistance to sulfate expansion. The data also suggests that the AFm phase is not prone to react and become an expansive AFt phase when exposed to an external source of sulfate. At equal w/c ratio, the use of FA and GGBS reduced further expansion by diluting the CSAB content but for a given strength CSABg concretes were better towards high strengths.

The addition of 50% v/v CDWA has only a minor detrimental effect in concrete durability. It was generally found to increase carbonation rates and chloride diffusion coefficients but to

reduce sulfate resistance. The former effects were due to increase in concrete absorptivity, whereas the latter one is assumed to be due to the higher gypsum addition used in reference CSABg concrete causing the formation of microcrakcs. However, for a given characteristic strength, the durability performance of the CSABg recycled aggregate concretes was improved mainly due to the lower w/c ratio required to give equal strength.

CHAPTER 7: COMPARATIVE PERFORMANCE AND PRACTICAL IMPLICATIONS

This section considers the practical implications associated with using the recycled aggregates and the various cementitious materials examined during the study in concrete production. This covers consistence, early strength, and a comparative approach of drying shrinkage, permeation and durability performance at equal strength. In addition, the environmental impact, associated with the use of the various material combinations in concrete (in terms of embodied CO_2) in relation to their associated performance is considered.

7.1 Consistence and SP dosage

The water compensation method selected in this study showed that conventional and recycled aggregate concretes can be produced in a similar way without the need of aggregate pre-saturation. This is of particular importance both because pre-wetting of recycled aggregates may not be practical due to the scarcity of the water and here is no requirement for concrete producers to modify production processes.

Given that the concrete mixes were designed for a fixed water content of 185/m^3 and an equal consistence i.e. BS EN 206-1 (BSI, 2013) consistence class S3 with a nominal slump between 100-150mm, the superplasticizer (SP) dosage was adjusted to achieve the required consistence. In all cases, the variation in the required SP dosage was low and within the recommended limits by the supplier. As the specific surface area of the CSABg cement is higher compared to CEM I and the sulfoaluminated phase is characterized by a high water demand (Winnefeld and Lothenbach, 2010), it was expected the resulting concretes to be more water demanding. In fact, the opposite case was found. For the same w/c ration and water content, the CSABg recycled aggregate mixes showed a higher fluidity from the equivalent CEM I recycled aggregate concretes and therefore less amount of superplasticizer was required for similar slumps. This was also noted in another study (Dachtar, 2004).

For the same w/c ratio, the workability of recycled aggregate concretes was generally found to improve with increasing the CDWA content, given that the water absorption of recycled aggregates was treated properly. This was the effect of the more spherical particles of CDWA

compared to reference granite aggregates, but the validation study showed that when gabbro or limestone aggregates are used as the reference material the resulting slumps were not very dissimilar. On the contrary, concretes incorporating EWA even at low replacement levels required higher superplasticizer dosages to achieve a similar slump within the target limits. Most likely this was due to the effect of increased presence of fines and therefore further processing is required before the material is used in concrete. Vibrating screens and air sifters can be used effectively to remove the excess of fines, as washing may not be practical and sustainable due to limited clean water sources.

The use of pozzolanic additions (i.e. FA and GGBS) was generally found to increase the workability of CEM I concretes and have no practical effect on workability of CSABg concretes. There was no significant difference in SP dosage requirements observed for the different binary cement combinations.

All CSABg-based concretes, however, lost initial workability rapidly due to the fast setting nature of CSABg cement. This coupled with the hot and dry climate of the Gulf region has the potential to accelarete further setting times and make concrete placement and compaction very problematic. The use of a retarding chemical admixture appears to be a must requirement for on-site applications of CSABg-based cement concretes.

7.2 **Strength development**

The strength in CSABg concretes was found to increase with reducing the w/c ratio, following a similar trend with PC concretes. However, the rate of strength gaining was much higher from that observed for CEM I concretes and all CSABg concretes reached at 3 days approximately 75-90% of their 28-day compressive strength. This was increased to about 90-100% at 7 days. The booming construction industry in Qatar can take advantage of this benefit and significantly reduce construction times, as less time is required for concrete curing.

The achieved compressive strength at 28 days was also much higher than that observed for CEM I concretes of equivalent w/c ratio and water content. In this regard, the content of CSABg cement could be significantly reduced to produce concretes of equivalent strength with Portland cement concretes. Given that the eCO_2 of CSAB is lower compared to PC, the environmental impact of concrete would be significantly reduced (this will be analysed in detail later). In terms

of likely cost savings, the price of CSAB cement per ton is higher compared to traditional CEM and taking into account the shipping cost of the material it may not reduce the cost per cubic meter of concrete.

CSABg concretes showed no further strength development after 28 days. Instead, all CSABg concretes with no recycled aggregates or pozzolanic additions experienced a significant strength loss after 28 days. This was in the range of 9-14% and 20-23% for 90 and 180 days, respectively. It is not clear why this occurred but raises serious concerns about the implementation of novel materials that do not have a long- established performance history and can potentially compromise structural integrity and durability of the structures.

In line with established behaviour, the compressive and flexural strength of concrete was gradually reduced with increasing the recycled aggregate content. This was noted at all w/c ratios and testing ages. It was also seen that for low substitution levels up to 30% v/v, the effect on 28-day compressive strength was minimal and within experimental variability. For the same w/c ratio and aggregate replacement level, there was no significant difference in the resulting strength of concrete incorporating CDWA or EWA of similar replacement level. To account for the strength loss of recycled aggregate concretes, it would be necessary in practice to reduce the w/c ratio of the mix by increasing the cement content or reducing the water content. This is a well demonstrated practice but above a certain level may result in not practical or sustainable concretes.

As expected, the pozzolanic reactivity of FA and GGBS within a CEM I-based system was higher and therefore concretes exhibited an on-going strength development, compared to CSABg-based concretes where minor or no strength gaining was noted at later ages. At an equal w/c ratio (i.e. 0.55), the use of 40%w/w FA or 50% w/w GGBS in blends with CEM I resulted in reduced 28-day compressive strengths by approximately 50% and 17%, respectively. The corresponding strength reductions for CSABg concretes incorporating 15% w/w FA or GGBS addition were 24.5% and 13%, respectively, relatively higher considering that the addition level was only 15% w/w. In all cases however, concretes incorporating GGBS in the cement blend performed better given the higher pozzolanicity of GGBS compared to FA. The reduced 28-compressive strength coupled with the minor contribution of FA and GGBS in long-term strength of CSABg concretes suggests that the use of pozzolanic additions above 15% by mass

of CSAB cement may lead to impractical mixes and also may compromise any eCO_2 savings resulted from the lower CSAB content required for equivalent strength with PC concretes.

7.3 **Engineering and durability performance based on equivalent strength**

A more practical and useful comparison of engineering and durability properties of concrete can be done on an equal strength basis. This is mainly because compressive strength of concrete is a mix design key criterion for the construction industry, although BS EN 206-1 (BSI, 2013) encourages concrete mix designs to be performance focused. Two concrete strengths equal to 40 N/mm^2 and 50 N/mm^2 were selected as the basis of the comparison because they satisfy specified minimum strength class limits for the majority of exposure classes recommended in BS 8500-1 (BSI, 2015), as discussed in Section 3.5.1. The results, which were obtained by interpolation from the data presented in Chapters 5 and 6, are given in Table 7.1 and Table 7.2.

7.3.1 Drying shrinkage

Dying shrinkage of both CEM I and CSABg concretes was found to increase with time and reduce with increasing strength. Although the trends where similar, CSABg concretes exhibited significantly lower shrinkages. As it can be seen in Table 7.1 and Table 7.2, the shrinkage values were reduced by approximately 50% at both test ages and strength classes. This is due to the expansive nature of CSABg cements achieving shrinkage compensation. Of additional importance is the lower shrinkage rate observed between 28 and 112 days. These effects constitute a great advantage of CSABg cements over PCs, particularly for concrete applications in hot weather, as would eliminate requirements for contraction joints and prolonged curing.

Given that CDWA and EWA have met the 0.075% aggregate drying shrinkage limit set in BS EN 12620 (BSI, 2002), drying shrinkage values of recycled aggregate concretes were comparable or slightly lower to those of reference concretes. This is due to the reduced w/c ratio required to achieve equivalent strength and offsetting the detrimental effect of recycled aggregates.

7.3.2 Absorption properties

In line with the expected behaviour, all concretes gave a decreasing water absorptivity at 28 days with increasing compressive strength. At all strength classes, CEM I-based concretes showed significantly reduced ISAT-10 values compared to CSABg-based concretes. The use of pozzolanic additions in both cementitious systems has resulted in comparable or slightly

improved permeation properties. The latter was mainly the effect of reduced w/c ratio used rather than any beneficial effect of FA or GGBS incorporation due to the slow rate of pozzolanic reactions.

These results come in disagreement with previous literature findings, where CSABg-based cements were reported to develop a dense and biomodal (disconnected and isolated pores) microstructure earlier than Portland cements (Glasser and Zhang, 2001; Garcia-Mate, et al., 2013; Telesca, et al., 2014) and the inclusion of pozzolanic materials (up to 15%w/w) in the CSAB blend to further reduces the total porosity (Zivica, 2000; Ioannou, et al., 2014). Consideration and further testing will be required to establish the absorption properties of CSABg concretes, as the PC database have shown that absorptivity, as a indication of permeability, and durability are linked together.

Use of recycled aggregates up to a level of 30% was not found to have any practical effect on the absorption properties of concretes. For higher replacement levels, however, the reduction in w/c ratio couldn't offset completely the increase in absorptivity and recycled aggregate concretes exhibited slightly increased water absorptivity, but this is not expected to alter much the durability of these concretes. When pozzolanic additions and recycled aggregates are used together in the mix design, a beneficial effect can only be found for mixes incorporating GGBS.

7.3.3 Carbonation

In comparison with CEM I-based concretes, the carbonation performance of all CSABg-based concretes was inferior both at 40 and 50 N/mm², with carbonation depths being double. This is mainly attributed to the low alkalinity of the CSABg cement, which was found to be around pH 12. The common practice of using the concrete compressive strength as an indirect indicator of its carbonation resistance does not apply when these two types of cement are compared on an equal strength basis. Even at very high strengths, where carbonation is usually considered negligible the use of CSABg cement in the concrete mixes resulted in high levels of carbonation. Therefore, considerable care has to be taken with the design of such mixes for steel reinforced concrete where carbonation-induced corrosion could occur. The low alkalinity of CSABg concrete, however, could be an advantage when glass fibres are used as reinforcement in concrete since these fibres are vulnerable to a high alkali and high pH environment.

As expected, the use of pozzolanic additions in blends with CEM I increased carbonation depths by approximately 25%. Neither the reduced w/c required for equivalent strength, nor the further reduction in concrete permeability due to pozzolanic reactions could compensate the reduction in lime available to maintain the pH level of concrete. Given the lower pozzolanic additions (i.e. 15% w/w) in CSABg concretes, CSAB/FA and CSAB/GGBS concretes showed similar or slightly improved carbonation resistance to reference CSABg concrete.

The carbonation resistance of recycled aggregate concrete was found to increase with replacement level, but this was compromised when pozzolanic additions were used in the same mix.

7.3.4 Chlorides ingress

The AFm/AFt ratio of CSABg cement was manipulated in the paste study to favour the former and enhance chloride resistance. Indeed, the data suggests an increased potential for binding chlorides which offsets the higher absorptivity of CSABg concretes and results in chloride diffusion coefficients reduced by approximately 50% compared to CEM I concretes. This also highlights that there is no additional risk of chloride-induced corrosion as shown for carbonation.

The use of pozzolanic additions within PC systems, and particularly that of GGBS, has further improved the chloride resistance of concrete because of the combined effect of reduced permeability and increased monosulfate content (AFm phase). In CSAB cementitious system, the addition of GGBS has some beneficial contribution, whereas that of FA resulted in similar diffusion coefficients. In any case, however, the chloride resistance of CEM I/FA and CEM I/GGBS concretes appeared to be significantly better.

Recycled concrete aggregates were not found to pose any further risk in terms of chloride ingress. In fact, the chloride diffusion coefficient of CEM I concretes is reduced with aggregate replacement level as a result of the higher cementitious contents increasing binding potential and reducing concrete permeability.

Table 7.1: Comparative behavior of 40 N/mm^2 strength concretes

Mix Name	w/c	Drying shrinkage, μstrain		ISAT-10min, (ml/m^2/s)	Chloride Diffusion Coefficient, $\times 10^{-12}$ m^{2}s^{-1}	Carbonation Depth, mm		60-week Sulfate Expansion, % of initial length	
		28-day	112-day			10 weeks	20 weeks	Immersed	wet & dry
40 N/mm^2 concrete									
CEM I GRA	0.65	670	860	0.49	26.0	14.5	20.0	0.060	0.090
CEM I GRA30CDWA	0.63	-	-	0.5	25.0	13.5	18.5	0.060	0.070
CEM I GRA50CDWA	0.61	725	885	0.55	23.0	12.0	18.0	0.055	0.080
CEM I GRA100CDWA	0.59	-	-	0.58	22.5	11.0	16.0	0.085	0.070
CEM I GRA15EXA	0.64	-	-	0.47	26.5	15.0	20.0	0.025	0.035
CEM I GRA30EXA	0.62	675	800	0.48	24.5	14.0	20.0	0.030	0.060
CEM I/FA GRA	0.45	-	-	0.44	6.0	18.5	25.0	0.015	0.015
CEM I/FA GRA30CDWA	0.44	-	-	0.49	6.5	18.0	25.0	0.015	0.020
CEM I/GGBS GRA	0.59	-	-	0.43	3.5	18.5	23.5	0.025	0.035
CEM I/GGBS GRA30CDWA	0.58	-	-	0.44	4.0	18.0	24.5	0.030	0.040
CSABg GRA	0.7	300	410	0.90	16.0	35.0	46.0	0.013	0.018
CSABg GRA50CDWA	0.63	250	360	0.85	15.0	27.0	41.0	0.012	0.016
CSABg/15FA GRA	0.61	-	-	0.64	14.0	29.0	43.0	0.023	0.024
CSABg/15GGBS GRA	0.67	-	-	0.62	10.0	36.0	49.0	0.011	0.01

Table 7.2: Comparative behaviour of 50 N/mm² strength concretes

Mix Name	w/c	Drying shrinkage, μstrain		ISAT-10min, (ml/m²/s)	Chloride Diffusion Coefficient, $\times 10^{-12}$ m²s⁻¹	Carbonation Depth, mm		40-week Sulfate Expansion, % of initial length	
		28-day	112-day			10 weeks	20 weeks	Immersed	wet & dry
50 N/mm²									
CEM I GRA	0.55	630	840	0.45	20.0	9.0	12.0	0.040	0.039
CEM I GRA30CDWA	0.53	-	-	0.44	19.0	7.0	10.0	0.031	0.036
CEM I/FA GRA	0.37	-	-	0.44	5.0	13.0	18.0	0.005	0.01
CEM I/FA GRA30CDWA	0.37	-	-	0.48	4.5	12.0	18.0	0.008	0.01
CEM I/GGBS GRA	0.47	-	-	0.37	2.5	14.0	19.0	0.018	0.02
CEM I/GGBS GRA30CDWA	0.45	-	-	0.38	3.0	14.0	17.0	0.02	0.02
CSABg GRA	0.62	290	390	0.76	12.0	26.0	36.0	0.015	0.019
CSABg/15FA GRA	0.52	-	-	0.52	12.5	23.0	33.0	0.02	0.022
CSABg/15GGBS GRA	0.57	-	-	0.61	9.0	28.0	38.0	0.026	0.029

7.3.5 Sulfate expansion

In comparison with CEM I concretes, CSABg concretes exhibited significantly reduced expansions showing a superior performance at both strength grades and testing conditions. The use of pozzolanic materials in the cement blends, particularly with CEM I, provided an additional degree of protection against sulfate expansion, by both diluting the amount of CAH and reducing the porosity of the concrete. Although the recorded expansion of CEM I/FA and CEM I/GGBS due to sulfate attack was significantly reduced, CSABg-based concretes were still found to outperform. This highlights their suitability of CSABg cements in aggressive environments with high presence of sulfates, including that of the Gulf region.

Similar to previous observations, the sulfate performance of recycled aggregate concretes was generally found to be comparable or slightly improved compared to that of reference concrete.

7.4 **Concrete mix embodied CO_2 based on equivalent strength**

The Mineral Product Association (Leese and Casey, 2015) have published tables with indicative production-related eCO_2 values for the main UK cementitious constituents of concrete and according to those the eCO_2 of CEM I, fly ash and GGBS are 913, 4 and 52 kg $CO_2/t_{material}$, respectively. As highlighted in literature, manufacturing of CSA cements is typically reported to result in CO_2 emissions savings of around 20-30% compared to Portland cement (CEM I), but reliable published figures were not found in the literature, possibly due to the high compositional variation of the material. Ioannou et al. (2015) estimated the eCO_2 of a CSA-belite cement to approximately 600 $kg/t_{material}$, based on the eCO_2 emissions associated with the individual mineralogical cement compounds, and this value was taken as reference in this study.

The eCO_2 of aggregate production is approximately 5 kg $CO_2/t_{material}$ (MPA, 2014). This is significantly lower compared to the eCO_2 of Portland cement, and although the volume of aggregates in the mix is much higher, the impact of aggregates on the total eCO_2 of concrete is usually insignificant.

It is important to note that CO_2 emissions associated with international and local transport are not accounted for in these numbers. The eCO_2 related to local transportation can be ignored, as most of the raw materials travelled by road almost equal distances, but the emissions

related to international shipping will be considered in this study. Therefore, the eCO_2 related to the sea freight was considered for raw materials originated from outside Qatar and the total eCO_2 of each individual raw material is given in Table 7.3. The CO_2 emissions related to concrete production and transportation were also not considered as it would be very similar for all concrete mixes.

Table 7.3: Total eCO_2 of raw materials

Raw materials	Origin	eCO_2	CO_2 emissions for sea freight			Total eCO_2,
		kg CO_2/t	km	kg CO_2/t/km *	kg CO_2/t	kg CO_2/t
CEM I	Qatar	930	-	-	-	930
Fly ash	India	4	≈2000	0.015	30	34
GGBS	Japan	67	≈10000	0.015	150	217
CSAB	China	600	≈10000	0.015	150	750
NA	UAE	5	≈2000	0.015	30	35
RA	Qatar	5	-	-	-	5

* Average value (WSC, 2018)

The estimated eCO_2 values for the test concretes are compared in Figure 7.1 and Figure 7.2, using the mix constituent proportions given in Table 3.3, Table 3.5, Table 3.6 and Table 3.7 and adjusted for an equal strength of 40 and 50 N/mm^2 using Table 7.1 sand Table 7.2.

The eCO_2 of CEM I-based concretes is heavily influenced by the eCO_2 of the Portland cement and as strength increases, the cementitious content also increases alongside with the eCO_2 of concrete. However, with the appropriate use of pozzolanic materials it is possible to achieve equivalent strength with lower eCO_2. Indeed, the use of FA (40% w/w) and GGBS (50% w/w) additions resulted in considerable CO_2 emissions reductions of about 12% and 30%, respectively.

When recycled aggregates used in the mix the total eCO_2 of the recycled aggregate concretes was comparable or lower to reference concrete. Although the eCO_2 associated with the cement content only was increased, almost proportional to replacement level, this more than compensated by the positive effect of recycled aggregate incorporation. The combined beneficial effect of using both pozzolanic additions and recycled aggregates was similar to the numbers reported above.

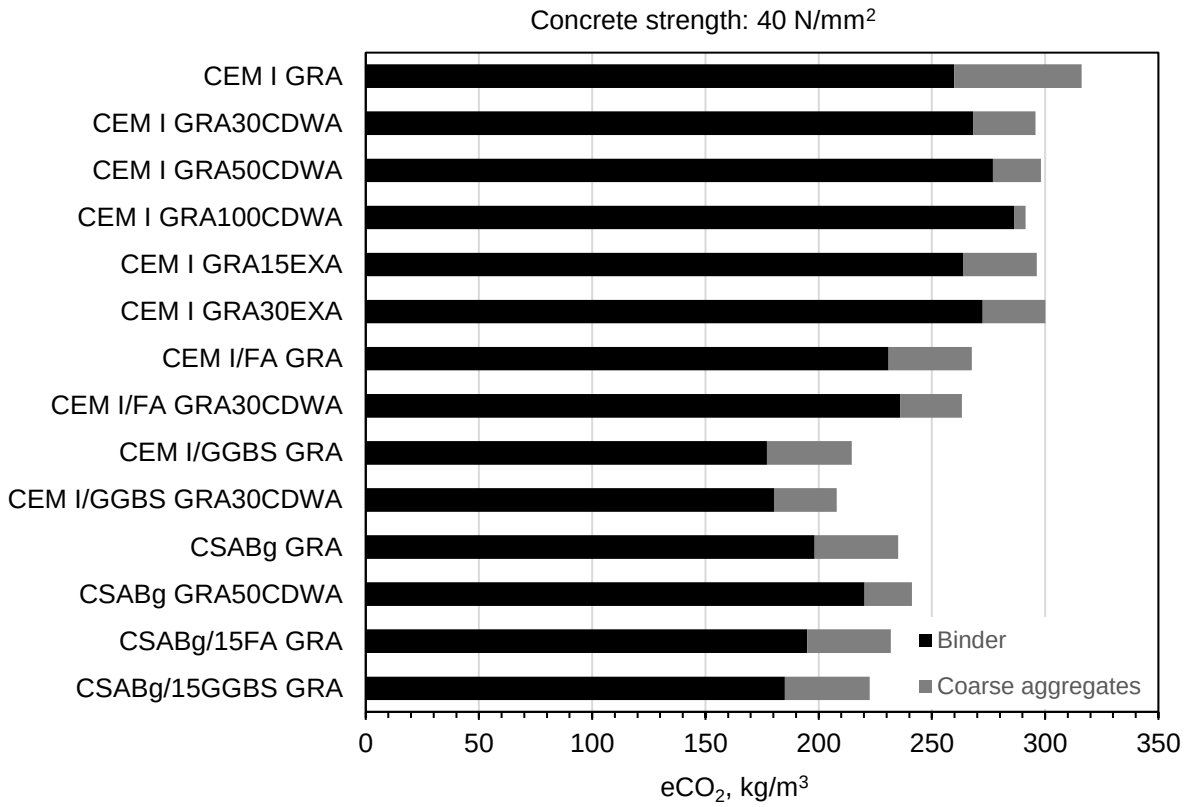


Figure 7.1: Comparative eCO_2 of equal strength concrete (40 N/mm²)

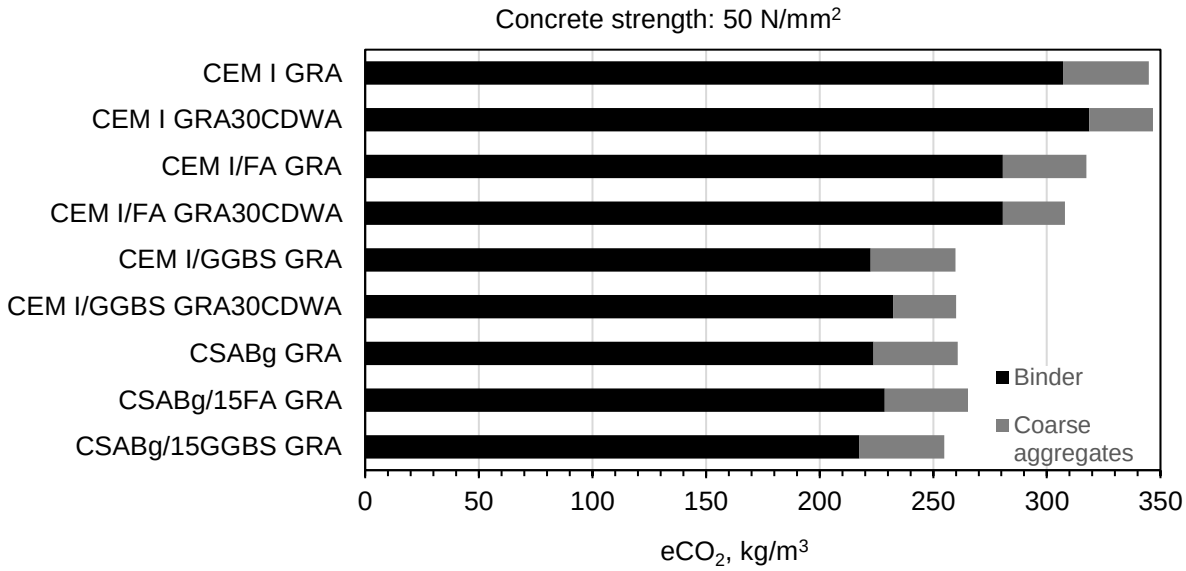


Figure 7.2: Comparative eCO_2 of equal strength concrete (50 N/mm²)

Compared to a CEM I concrete of equivalent 40 N/mm² cube strength, the potential eCO_2 savings related to the use of CSAB cement only were about 15%. The use of 15% pozzolanic additions has further reduced the total eCO_2 by approx. 14%, but still CEM I/GGBS mix did show lower eCO_2 value. At 50 N/mm² equivalent cube strength the savings were increased to

approx. 25%, but pozzolanic additions did not contribute to any further substantial carbon savings and thus their environment impact was similar to that of CEM I/GGBS concrete. This was due to effect pozzolanic material incorporation and particularly that of FA, in relation to compressive strength of concrete, which required the total cement content in CSABg/15FA and CSABg/15GGBS concretes to be increased by approximately 20% and 10%, respectively, to achieve the same strength.

7.5 **Conclusions**

The comparison described in the previous sections has demonstrated that there is considerable potential for the use of recycled aggregates in Qatar. On an equal strength basis, the engineering and durability performance of all CEM I recycled concretes aggregate concretes was generally similar to that of reference concretes with only small differences in the resulting data. This coupled with the generally reduced eCO_2 of these concretes, highlights the suitability of recycled aggregates and particularly that of CDWA at replacement levels higher than the 20% by mass currently allowed by the QCS 5-2 (QS, 2014). As expected, the additional use of pozzolanic materials in the mix design and particularly GGBS has further improved the durability performance, with the exception of carbonation resistance, and reduced substantially the associated eCO_2.

In comparison with a CEM I concrete, CSABg concrete exhibited rapid strength development, lower drying shrinkage, lower chloride diffusion coefficient, superior resistance to sulfate attack and substantial carbon savings. On the negative side were handling, major strength losses, increased absorptivity and reduced resistance to carbonation. Although, the use of a suitable retarding agent could improve material handling, and pozzolanic additions could eliminate strength losses and reduce permeability, the carbonation rates would still be high. There is, therefore, some potential for the use of CSABg cements in selective concrete applications, but further material optimisation and testing, including site trials, are required before the construction industry make a partial shift away from traditional CEM I/FA or CEM I/GGBS mixes.

CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

The purpose of this project was to develop sustainable and environmental friendly concretes by utilising locally produced recycled aggregates, a novel low carbon non-Portland cement and pozzolanic additions. This involved firstly, characterizing the materials to ensure suitability for use in concrete. Following this, concrete tests in a laboratory environment were carried out to determine the effect of these materials solely and in combinations on fresh, engineering and durability properties of concrete. Finally, the performance, environmental impact and practical implications of the test concretes were compared on an equal strength basis to identify potential material combinations that meet both durability and sustainability criteria and can be used by the construction industry in Qatar.

8.1 Recycled aggregate characterisation

Given the need to reduce imports of primary aggregates from UAE, locally produced coarse and fine recycled aggregates from construction and demolitions activities (CDWA) and excavation wastes (EWA) were physically and chemically characterized using the test methods outlined in Section 3.3. Their properties were then compared to those of natural aggregates imported from UAE and to the limits set by the QCS 5-2 (QS, 2014) to establish their suitability for use in concrete. The findings of this part of the study are summarized here:

- Based on composition requirements, both coarse CDWA and EWA can be classified as crushed concrete aggregates (CCA).
- Not all the coarse CDWA and EWA test samples met the grading requirements. Similarly, the fine fraction of CDWA appeared to be quite coarser, whereas the gradation of fine EWA was within the specified limits.
- The fines contents of coarse and fine CDWA were generally within the specified limits. On the contrary, all EWA samples but mainly the 4/10 and 0/4 sample sizes had fines contents significantly greater than the allowable one. The high presence of fines within the EWA may result to consistence and shrinkage problems when these aggregates are used in concrete.

- The particle density of CDWA and EWA was lower compared to natural aggregates due to the porous nature of adhered cement paste and/or porosity of parent aggregate, but still suitable for normal weight concrete applications.
- The water absorption of coarse CDWA and EWA was relatively higher than the 3% conformity limit specified in QCS 5-2 (QS, 2014). In case of fine CDWA and EWA the water absorption was found greater than 14%, which is significantly higher than the 4% limit, and thus making the fine fraction unsuitable for any concrete applications. A good correlation was also found between the water absorption and particle density (on SSD state) of test aggregates.
- The flakiness index (FI) of coarse CDWA and EWA was significantly below the 35% limit. In comparison between CDWA and EWA, the CDWA had more round shaped particles. The aggregate flakiness index was also found to reduce with size fraction.
- All coarse CDWA and EWA samples met the limits for resistance to fragmentation, drying shrinkage and expansion due to alkali aggregate reaction, with the CDWA showing relatively better performance.
- Coarse CDWA and EWA have generally failed to comply with the maximum percentage for acid-soluble chloride and sulfate contents. Coarse EWA has also failed to meet the limit for water-soluble sulfate content.
- Fine CDWA and EWA have failed to comply with the maximum percentage for acid-soluble chloride and sulfate contents, as well that for water-soluble sulfates.
- From XRD, it was revealed that CDWA comprised from belite, portlandite, gypsum, ettringite, calcite, dolomite, quartz, albite and kaolinite, whereas EWA from quartz, calcite, dolomite, albite, gypsum and clay minerals. The presence of gypsum and dolomite are typical in the region.
- By nature recycled aggregates are very heterogeneous materials and indeed the properties of both CDWA and EWA were found to vary a lot with size fraction and between samples of the same designation.
- Imported coarse gabbro and limestone aggregates have passed all the physical, mechanical and chemical limits, and in comparison with recycled aggregates they had lower water absorption, lower fines content, higher densities, higher resistance to fragmentation, comparable FI, lower drying shrinkage and lower percentages of acid-

soluble chlorides and sulfates. Similarly, the washed sand was generally conforming with all the limits, with the exception of acid-soluble sulfate content.

Overall the characteristics of coarse and fine CDWA and EWA were inferior to those of imported coarse aggregates (gabbro and limestone) and local washed sand. In addition to that, most recycled aggregate samples failed to meet all the requirements specified in QCS 5-2 (QS, 2014). However, the properties of all CDWA and EWA were generally conforming to the broader limits of BS EN 12620 (BSI, 2002). Based on the above findings the coarse CDWA and EWA could be suitable for use in concrete but further processing is required in order to fully conform with the current version of QCS 5-2 (QS, 2014). Recycled aggregate sands were highly absorptive and rich in sulfates and it is very unlikely to be used in structural or non-structural concrete applications.

8.2 **CSAB cement characterisation**

Given the need for the utilization of alternative cementitious systems to reduce the environmental impact of concrete associated with Portland cement use, an alternative cementitious system of lower eCO_2, that of calcium sulfoaluminate (CSAB) cement, was considered in this study. The composition of CSA-based cements varied considerably within literature and its properties were reported to be much influenced by the amount of added calcium sulfate. According to stoichiometric equations, a gypsum content of 23% by mass of CSAB is required to produce ettringite exclusively. However, the properties of this CSAB cement were investigated in relation to different additions of gypsum ranging from 0 to 23% w/w. The following conclusions are made according to the results of this paste/mortar study.

- The ettringite formation increased with increasing the gypsum addition. Thus, the amount of added gypsum can be used to change the ratio of AFm/AFt hydrates in mature CSAB cement paste.
- The MIP results showed that the permeability of CSAB cement paste, as defined by total porosity and interconnectivity of said porosity, reduces with increasing gypsum content.
- At very low gypsum contents (0-1% by mass of CSAB), the CSAB cement pastes had initial and final setting times comparable to that of Portland cement. For higher gypsum additions, however, the initial set of the 0.3 w/c ratio paste occurred at approximately after 20 mins, with the final set observed within the next 10 minutes. The increase of w/c

ratio from 0.3 to 0.5 had some positive influence, but still the setting times were quite short, typically in the range of 60 to 75 minutes.

- The alkalinity of the CSAB cement paste at all investigated gypsum contents was around pH 12.0, which is much lower than PC, but slightly above the critical pH value of 11.5 required for full steel reinforcement passivation.
- The expansion of the CSAB cement mortar was found to increase with gypsum additions up to 5% w/w and then to reduce again for higher contents. In general, the expansion patterns of CSAB cement mortars having gypsum additions less than 2% or above 9% w/w were very low and similar to that of a reference PC sample.
- All CSAB cement mortars achieved maximum strength within the first 3 days, but no further strength gaining was noticed afterwards. Similar to previous observations, gypsum additions up to 2% w/w helped the CSAB system to achieve its potential maximum strength, with higher additions resulting in reduced strengths by approx. 40%. No correlation was found between the measured total porosity of samples and the strengths observed, with higher porosity corresponding to lower strengths.
- The resistance of CSAB cements to chloride ingress should be determined using test methods of sufficiently long duration to allow binding to form, as rapid testing was found to be unrepresentative, if highly conservative. The binding capacity of this CSAB cement was appeared to be inversely proportional to the total gypsum content.

Overall, the gypsum content does influence the properties of the CSAB cement in a different way and appears that the optimum for full ye'elimite hydration or porosity is not coincident for strength or chloride ingress. The present data suggests a gypsum addition of no more than 2% by mass of the CSAB cement to achieve maximum potential strength, enhance chloride resistance, reduce expansion potential and prevent rapid setting.

8.3 **CEM I recycled aggregate concretes**

Having established the characteristics of CDWA and EWA, this part of the study considered their effect on fresh, engineering and durability properties of concrete. The concretes considered are those described in Section 3.5.1 and tested as outlined in Section 3.5.4 under controlled laboratory environments. The concrete properties were compared at both equal w/c ratio and strength basis. The findings of this part of the study are summarized below:

- The plastic density of all recycled aggregate concretes was reduced with increasing replacement level, owing to the lower densities of coarse recycled aggregates.
- When the FI values of coarse CDWA and natural aggregates were comparable, no effect on workability of the concrete was noted. On the contrary, coarse EWA reduced drastically the consistence of the concrete, even at small additions, due to the increased presence of fines.
- For a given w/c ratio the test results showed that CDWA and EWA up to a 30% v/v replacement level had a negligible effect on the compressive and flexural strength of concrete, but thereafter these reduce with increase in recycled aggregate content.
- No differences were noted in the fracture pattern/surfaces of concretes containing or not recycled aggregates.
- To take account of the effects of recycled aggregates on compressive strength, a simple adjustment of w/c ratio would be required.
- The general trends observed indicate that for a given w/c ratio the use of CDWA and EWA at replacement levels up to 30% v/v had no or little detrimental effect on drying shrinkage, water absorptivity of concrete, carbonation resistance, chloride ingress and sulfate resistance. At higher replacement levels, the performance was generally found to be inferior.
- Compressive strength was found to correlate well with all durability properties, namely water absorptivity, carbonation and chloride ingress.
- All recycled aggregate concrete mixes, for a given strength, were found to possess durability properties comparable or improved to the reference natural aggregate concrete mix. This effect was due to the lower w/c ratio required to achieve equal strength. The eCO_2 of these concretes was also found lower. Although, the eCO_2 associated with the cement content only was increased, almost proportional to replacement level, this more than compensated by the positive effect of recycled aggregate incorporation.
- For the same strength grade, the addition of pozzolanic materials and particularly GGBS reduced permeability, enhanced significantly the resistance to chlorides and sulfates and resulted in considerably higher carbon savings. Carbonation depths were increased though.

The data presented here has demonstrated the suitability of locally produced coarse recycled aggregates for use in concrete applications under a range of exposure conditions. Indeed, the results have shown that up to 30% v/v coarse CDWA or EWA can be used at equal w/c ratio without affecting much engineering and durability properties. From an equivalent strength basis though, it is possible to achieve higher implementation levels of recycled aggregates without compromising engineering, durability and environmental performance. This should encourage even more engineers and contractors to adopt a more relaxed and rational attitude towards the use of recycled materials in concrete production. In order to ensure the required durability in the aggressive environment of Qatar and further enhance sustainability of these mixes, the use of pozzolanic materials in combination with recycled aggregates is highly recommended.

8.4 CSABg concretes with or no recycled aggregates and pozzolanic additions

This concrete work aimed to explore the potential of CSAB cement as the main building material for structural grade concrete and identify any limitations of the material in this context. The concretes considered are those described in Section 3.5.1 and tested as outlined in Section 3.5.4 under controlled laboratory environments. The concrete properties were compared at both equal w/c ratio and strength basis. Comparison were also made with CEM I-based concretes. The findings of from these can be summarized as follow:

- Concretes mixes using CSABg, CSABg/15FA or CSABg/15GGBS cements all lost initial workability rapidly. Although there was sufficient time prior to setting to allow laboratory test samples to be compacted and finished, for on-site applications and concreting in hot weather a retarder would be required.
- Although, as well documented, the early strength development for CSABg-based concretes is faster than that of CEM I-based concretes, between 28 and 180 days of standard water-curing CSABg concretes lost up to 23% of their 28-day strength. Although it was not clear what the cause of this was, it is likely that internal cracking due to volume instability was a primary issue. The addition of pozzolanic materials counteracted the strength loss and thus CSABg/15FA and CSABg/15GGBS concretes maintained their 28-day strength almost constant with time. As expected, the pozzolanic reactivity of fly ash and GGBS within a PC-based system was higher and therefore CEM I/FA and CEM I/GGBS concrete exhibited on-going strength development.

- The flexure to compressive strength ratio of all CSABg concretes was in the range of 0.06 to 0.09, slightly lower from that observed for CEM I concretes i.e. 0.1 to 0.13.
- For a given w/c ratio or strength, all CSABg concretes experienced very low drying shrinkages compared to equivalent CEM I concretes, even though the water losses were comparable. This was attributed to the expansive nature of CSABg cement achieving shrinkage compensation.
- Similar to conventional Portland cement technology, the permeation and durability properties of CSABg concretes were strongly associated with the w/c ratio of the mix.
- In comparison with CEM I concretes of equivalent w/c ratio or strength, CSABg concretes had lower chloride diffusion coefficients, higher resistance to sulfate expansion, but higher absorptivity and significantly lower carbonation resistance.
- The high carbonation rates are related to the reduced amounts of alkali hydroxides present. This suggests that the current practice of using the compressive strength in Portland cement concrete as an indirect indicator of its carbonation resistance would be misleading in case of CSABg concretes.
- The AFm/AFt ratio was manipulated in this study to favour the former and the data suggests an increased potential for binding chlorides which offset the higher absorptivity of CSABg mixes.
- Given the AFm content and higher absorptivity of these mixes a decreased resistance to sulfate attack was expected. The data instead shows that the AFm phase is not prone to react and become an expansive AFt phase when exposed to an external source of sulfate.
- For a given w/c ratio, CSABg cement blends with pozzolanic additions resulted in a comparable or inferior durability performance to that of CSABg-only concrete. However, for a given characteristic strength, the durability properties were improved mainly due to the lower w/c ratio required to give equal strength. In comparison with a typical CEM I/FA or CEM I/GGBS concretes, the durability performance of all CSABg-based concretes was inferior at both equal w/c ratios and strength basis. In particular, CSABg-based concretes had higher ISAT-10 values, lower carbonation resistance, higher chloride diffusion coefficients, although sulfate resistance was comparable.
- The effect of 50% v/v coarse CDWA addition on properties of CSABg concrete followed similar trends to those described in Section 8.3.

- Based on eCO_2 data, all CSABg-based mixes did show eCO_2 values reduced by approx. 25% compared to equal strength CEM I concretes. In comparison with typical CEM I/FA or CEM I/GGBS, however, the carbon savings where comparable.

Under current design procedures, some potentially suitable applications for CSABg-based concretes include unreinforced concrete applications and aggressive environments. In these cases, it is likely CSABg-based concretes to satisfy design criteria and yield carbon savings lower than CEM I concretes. However, given the superior durability performance and comparable eCO_2 of CEM I/FA or CEM I/GGBS mixes, a partial shift away from these concretes would be unlikely due to handling and durability issues of the CSABg-based concretes.

8.5 **Recommendations for further study**

The aim of this project is to support the rapidly developing infrastructure industry in Qatar to overcome construction aggregate shortages and meet its sustainability targets by implementing locally produced recycled aggregates and low carbon non-Portland cements in construction. The laboratory testing has demonstrated the potential of recycled aggregates as a valuable source of construction aggregates in Qatar and CSAB cements to make a significant contribution to reduce eCO_2 of concretes. Based on that findings, recommendations for further work/actions include the following:

8.5.1 Recycled aggregates

Areas of research

- The high water absorption values of CDWA and EWA are currently prohibitive for use in structural concrete. There are certain hydrophobic type chemical admixtures that could provide a hydrophobic lining along the external surface of the aggregate particle; thus, it would be interesting to investigate to what extend these admixtures can reduce the water absorptivity of these recycled aggregates.
- This study has highlighted in general the environmental and economic benefits of using locally produced recycled aggregates rather than importing primary aggregates, but a more comprehensive study showing the actual reductions in greenhouse gas emissions and cost savings is required to change behaviour in industry.

Actions required

- If a Code of Practice is developed and implemented for construction and demolition waste it might improve the quality and consistency of material arriving at the recycling plant.
- It is required recycled aggregate producers to be certified as part of the Qatar Construction Specification certification scheme.

8.5.2 Calcium sulfoaluminate-belite (CSAB) cements

Areas of research

- It has been shown that the gypsum addition has a significant effect on the achieved compressive strength of mortar. Following that, it is suggested to investigate the role of microstructure and hydration products in strength development of CSAB cements.
- This study has focused upon a single durability issue, i.e. chloride ingress. An assessment of the carbonation potential of CSA-based cements will be necessary to ensuring long term steel passivation stability where structures are not fully saturated and binding is compromised by the resulting lower pore alkalinity. Similarly, the sulfate resistance has to be established, possibly in a parametric approach, as the amount of added gypsum can influence the AFm/AFt ratio and hence the amount and type of hydrates that are available to react with sulfates from external sources and cause expansion and/or degradation.

8.5.3 Recycled aggregates concrete

Areas of research

- Given the increasing importance of recycling aggregate, and the relevance of embracing a performance based approach to the specification of concrete, it is recommended that further work, similar to that reported here, be undertaken so that the engineering and durability database for the recycled aggregate concretes can be further strengthened for its adoption in practice.
- Elastic modulus is a fundamental parameter in designing concrete structures. Various empirical, practical or universal equations have been proposed for predicting the modulus of elasticity when concrete includes coarse natural or recycled aggregates and mineral additions. However, such equations are not applicable to the whole data for recycled aggregate concrete because its mechanical properties depend greatly on both the properties of recycled aggregates and the ratio of the mix proportions within such

concrete. Thus, it is recommended to establish the effect of various contents of CDWA and EWA to the modulus of elasticity of structural grade concretes.

Actions required

- Laboratory findings will require validation from site trials and field data.
- Qatar Construction Specifications should be reviewed and updated to enable the wider use of recycled aggregates in construction, and particularly in concrete production.
- Guidance should be provided for designers, contractors and producers on how to use recycled aggregates in concrete, where some changes to standard mix designs will be required.

8.5.4 CSAB-based concretes

Areas of research

- Plastic shrinkage is also important as can lead to excessive surface cracking. It is recommended further concrete testing be carried out to establish the plastic shrinkage behaviour of CSAB concrete and see if it behaves differently.
- When concrete is loaded, it undergoes elastic and inelastic deformations. Modulus of elasticity is linked to the elastic behaviour of concrete, whereas creep refers to inelastic deformation. Both properties are influenced by numerous factors, including, age and strength of concrete, type and density of aggregates, type of cementitious materials, environmental conditions, magnitude of the sustained loading, stress period, etc. Given the lack of sufficient experimental data, it would be interesting to establish the modulus of elasticity and investigate the long-term deformation of concretes produced with CSAB cements.
- The long-term durability of CSAB concretes is of particular interest and the current study has demonstrated using an accelerated testing protocol that considerable care has to be taken with the design of such mixes for steel reinforced concrete where carbonation-induced corrosion could occur. It is recommended that carbonation testing be carried out using normal exposure conditions i.e. 0.035% instead of 4% CO_2, to see if the trends are similar

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